GEOTECHNICAL ENGINEERING INVESTIGATION

NIPOMO CREEK BRIDGE AT WILLOW ROAD
COUNTY OF SAN LUIS OBISPO, CALIFORNIA

For

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INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed Nipomo Creek Bridge for Willow Road Extension in Nipomo, an unincorporated area located in southwestern San Luis Obispo County, California. Our work was performed generally in accordance with the scope of work as per our agreement. The location of the site and its vicinity are shown on the Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. These recommendations should not be used for bidding purposes or directly for construction cost estimates.

PROPOSED CONSTRUCTION

Based on the plans provided by Rajappan & Meyer Consulting Engineers Inc., the proposed project consists of extension of the existing Willow Road to Route 101 in Nipomo, California. As part of this project, Willow Road will extend eastward towards Route 101 from where it currently intersects Pomeroy Road. The road extension will continue with an interchange at Route 101, an undercrossing under the existing Route 101, a bridge crossing over Nipomo Creek, and ends by joining at grade with the existing North Thompson Avenue. This report will cover the results of our geotechnical engineering investigation for the proposed bridge crossing over Nipomo Creek northeast of Route 101.

The proposed Nipomo Creek Bridge is a 5-span slab bridge supported on pile extensions. The proposed bridge is approximately 189 feet in length and 43 feet in width. The approach roadway requires up to approximately 10 feet of new embankments.

Our recommendations presented in this report are based on the above information. Any major deviations should be reported to our office for further consideration.

PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to evaluate their engineering properties, and to provide recommendations for foundation support of the proposed Nipomo Creek Bridge.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature, obtain representative soil samples and log soil materials encountered in three drilled borings, laboratory testing of the collected samples, engineering analysis of the field and laboratory data, and preparation of this report.

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter unforeseen variations in the soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional geotechnical engineering services to attain a properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

SITE CONDITIONS

The project site is located at approximately 500 feet northeast of the intersection of the proposed Willow Road extension and the existing Route 101 in Nipomo, California. Nipomo Creek trends approximately parallel to Route 101 at the site. Based on the topographic plans provided to us, the site vicinity is generally sloping northeastward from Route 101 with approximate gradient of 7%. An existing nursery is located on the east of Route 101. Moderate vegetation, shrubs, and trees cover the general areas of the project site. The areas north and east of the site are currently occupied by open fields. The existing grade near the Nipomo Creek bed is at approx. Elev. 332 feet.

FIELD EXPLORATION

Based on the preliminary plans, discussions with the designers, and readily available geotechnical data in the area, three (3) new exploratory borings were drilled at selected locations to maximum depths of 90 feet below the existing ground surface. The approximate locations of the borings are shown on the Site Plan, Plate 2.

Two test borings (07-NP-1 and 07-NP-2) were advanced using a truck-mounted drill rig using rotary wash and one boring (07-B-13) was drilled using hollow stem auger drilling method. Selected samples were obtained from 1.4-inch I.D. (Standard Penetration) and 2.5-inch I.D. (Modified California) samplers at various depths. The samplers were driven into subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. (When correlating standard penetration data, the blow counts for the Modified California Sampler may be converted to equivalent SPT-N value by multiplying with a factor of 0.65). The samples were sealed and transported to our laboratory for further evaluation and testing. The field investigation was conducted under the supervision of our field engineer who logged the test borings and prepared the samples for subsequent laboratory testing and evaluation.

The Log of Test Borings (LOTB) summarizes the findings from the exploration and is presented in Appendix A. The boring logs presented were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs. The abrupt stratum changes shown on these logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations.

Laboratory tests were performed on selected samples obtained during field exploration to determine the physical and engineering properties of the subsoils. Laboratory data of moisture contents, dry density, Atterberg Limits, grain size distribution, and unconfined compression tests are presented on the LOTB. The detail results of laboratory tests are presented in Appendix B.

Geophysical survey was conducted by NORCAL Geophysical Consultants, Inc. in March 2007 to investigate the possible subsurface lateral lithologic or structural variations that may be associated with recent or older faulting at this site. The detail report is attached in Appendix C.

GEOLOGY

General geologic features pertaining to the site were evaluated with reference to the geology map titled "Generalized Geology of the Arroyo Grande-Nipomo Mesa Area" by the Department of Water Resources, Southern District, 2002. The accompanied geologic section C-C' was also referenced in our evaluation. Based on the map and the section, the subject site is generally underlain by older alluvial deposits (Qoal). The geologic map and the geologic section C-C' are shown on Plates 3A and 3B, respectively. This map and the geologic section also indicated the close proximity of the Wilmar Avenue Fault to the project site. Further discussions of the fault and relevant seismic issues are covered in a later section of this report.

SUBSURFACE CONDITIONS

Based on the investigation and readily available data, the subsoils generally consist of stiff to hard clay with interbedded layers of medium to very dense silty sand and medium dense gravel. Groundwater was encountered during field exploration in January 2007 at approximately 47 feet depth (approx. Elev. 286 ft) in Borings 07-B-13. However, groundwater level may vary with the passage of time due to seasonal rainfall, creek water level, surface and subsurface flows, ground surface run-off and other factors that may not be present at the time of investigation.

Detailed descriptions of the materials encountered in each of the exploratory borings are presented on the LOTB, Appendix A. It should be noted that these descriptions and related information depict subsurface conditions only at the locations indicated on the LOTB and on particular date noted. Because of the variability from place to place within soils in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. Also, the passage of time may result in a change in soil conditions at these locations due to environmental changes.

EARTHQUAKE CONSIDERATIONS

Seismic Sources

The project site is located in a seismically active part of central coastal California. Many faults in the San Luis Obispo area are capable of producing earthquakes that may cause strong ground shaking at the site. Maximum credible earthquake magnitudes for some of the major faults in the area as determined by Mualchin (1996) are summarized below. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure. It should be noted that this discussion pertains to faults recognized in the Caltrans published data of Mualchin (1996).

EARTHQUAKE DATA

Fault	Type	Estimated Distance From Project Site (km)	Maximum Credible Earthquake	Anticipated Peak Bedrock Acceleration
SMF – Santa Maria River-Foxen Canyon	Reverse (including thrust)	< 1	6.5	0.7
OCO – Oceano	Reverse (including thrust)	4.3	6.0	0.4

Active faults in the vicinity include the Santa Maria River-Foxen Canyon Fault (reverse fault, Mw = 6.5) located within 1 km from the site, and the Oceano Fault (reverse fault, Mw = 6.0) located at approximately 4.3 km from the site. Based on the seismic hazard map prepared by Mualchin (1996) and attenuation relationship proposed by Sadigh, et al (1997), a Peak Bedrock Acceleration of 0.7g is anticipated at the site. The fault map for the project vicinity is shown on the Fault Map, Plate 4. This map, however, does not include the Wilmar Avenue Fault. It is discussed in a separate section.

Seismic Hazards/Liquefaction Potential

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking, and liquefaction.

Surface Fault Rupture

According to the County of San Luis Obispo Local Hazard Mitigation Plan (LHMP), the Santa Maria River and Foxen Canyon faults are buried northwest-striking reverse faults. The fault trace is buried and is inferred to parallel the Santa Maria River and U.S. Highway 101. The fault zone is mapped as extending into the southern segment of the Wilmar Avenue fault zone and appears to coincide with Nipomo Creek, which is within the project vicinity. The geologic section (profile C-C' as shown on the geologic map) is attached on Plate 3B and it shows the relative locations of Wilmar Avenue Fault/Nipomo Creek and Route 101.

The northwest-trending Wilmar Avenue Fault is a late Quaternary reverse fault which is exposed only at a sea cliff in Pismo Beach. Elevated marine terraces on both sides of the fault suggest that the Wilmar Avenue Fault is not the primary fault that separates the uplifting San Luis/Pismo structural block from the subsiding Santa Maria Valley block. Along the southerly segment, the fault is inferred by the alignment of subtle geomorphic and geologic features, including a straight segment of Nipomo Creek. For this section of the fault, there is no surface expression. Therefore, fault trenching was not considered as an option.

The Wilmar Avenue Fault is considered as potentially active by the County's LHMP. According to USGS Bulletin 1995-AA, the Wilmar Avenue Fault is a moderately to steeply northeast-dipping reverse fault with late Quaternary slip rate of 0.01 to 0.1 mm/yr. According to USGS Bulletin 1995-BB, the post early/late Pliocene vertical slip rates across the Hosgri Fault Zone (which included the Wilmar Avenue Fault in the San Luis/Pismo reach) range from 0.1 to 0.4 millimeters per year, but may be as high as 0.44-millimeters per year if the rate of right-lateral slip along the fault is greater than 1 millimeter per year.

In order to identify the presence of the Wilmar Avenue Fault, a geophysical/resistivity survey was conducted in March 2007 by NORCAL Geophysical Consultants in the immediate area of the site. The report concludes that:

> *"In comparing the two profiles, a major, common subsurface feature appears as a near vertical lateral resistivity change approximately 50 feet west of the existing stream channel. This vertical boundary appears to separate primarily high*

> *resistivity zone interpreted as bedrock on the southwest from low resistivity zone as alluvium on the northeast given the limited geologic information available. This discontinuity can be explained by the presence of an interpreted near vertical fault boundary. This fault trend is northwest coincident with the course of Nipomo Creek and parallel to US Route 101. Our results show a subsurface structure that is highly suggestive of a fault."*

Based on the results of the geophysical survey, the fault trace was estimated to be located within the Creek and appeared to be approximately between the proposed Abutment 1 and Pier 2. However, we understand that the geophysical survey was conducted based on existing site landmarks (i.e. gates and fences) and measurements were taken on undulating ground profiles. Therefore, some minor discrepancies should be expected. It should be recognized that NORCAL Geophysical has defined the primary "boundary" as the fault trace. Generally, additional traces can develop along these features. A copy of the geophysical survey result is attached in Appendix C.

Based on the available data, the fault cannot be located at the creek surface. According to the conclusions from NORCAL study, it is suggested that the fault is within the creek and probably between the Abutment 1 and Pier 2. However, more shears could be present at depth. Regardless of these findings, the anticipated vertical movement along this fault (per USGS Bulletin 1995-BB) is approx. 12 mm (less than ½ inch) over a period of 30 years (0.4 mm/yr slip rate). Therefore, it is recommended to locate the supports with a minimum of 20-foot offset from the mapped fault, and design the structure to accommodate the anticipated fault slip over its design life. Regular maintenance of bridge approach should also be expected.

Ground Shaking

Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered moderate to high.

Liquefaction Potential

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Clays are generally not susceptible to

liquefaction.

Based on the current investigation, the groundwater level in the project vicinity is at about 47 feet (Elev. 286 ft) below existing grade. In general, the effect of liquefaction in the immediate vicinity of the site may be considered moderate.

FINDINGS AND RECOMMENDATIONS

General

Based on the findings of our investigation, it is our opinion that the site is feasible for the planned project provided the recommendations presented in this report are incorporated into the final design and construction.

This report was prepared specifically for the proposed structure as presented in the general plan provided to us. Normal construction procedures were assumed throughout our analysis and represent one of the basis of recommendations presented herein. Our design criteria have been based upon the materials encountered on the site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

Foundations

Based on the Nipomo Creek Bridge General Plan, provided to us from Rajappan & Meyer, the bridge is a 5-span CIP/PS slab structure supported on pile extensions. Based on the general plan, the two abutments and the approach roadway will require up to 10 feet of new embankments.

It is planned to use 14-inch square PC/PS concrete piles at the abutments and 18-inch octagonal PC/PS concrete piles at the piers, respectively.

Based on the information provided by the designer, the abutment footings are planned at Elev. 330.0 (Abut 1) and Elev. 330.75 (Abut 5). The piles will connect directly to the bottom of the bridge deck. The 14-inch square PC/PS concrete piles are planned for design load up to 100

Kips. The 18-inch octagonal PC/PS concrete piles are planned for design load of up to 160 Kips. The recommended specified pile tip elevations are shown in the pile data table below.

PILE DATA TABLE

Location	Pile Type	Design Loading (Service, kips)	Nominal Resistance (kips)		Design Tip Elev. (ft)	Specified Tip Elev. (ft)
			Compression	Tension		
Abut 1	14" sq. PC/PS	100	200	0	300 (1) & (2)	300
Pier 2	18" oct. PC/PS	160	320	0	275 (1); 295 (2)	275
Pier 3	18" oct. PC/PS	130	260	0	282 (1); 295 (2)	282
Pier 4	18" oct. PC/PS	135	270	0	282 (1); 295 (2)	282
Pier 5	18" oct. PC/PS	125	250	0	282 (1); 295 (2)	282
Abut 6	14" sq. PC/PS	90	180	0	300 (1) & (2)	300

*Design tip elevations are controlled by the following demands: (1) Compression, (2) Lateral Load

As noted in the "Potential Fault Creep/Offset" section, the fault trace appears to be near vertical. However, some inclination is indicated. The pile layout and the length should be designed to avoid intersecting the hypothetical line that defines the fault trace.

Driven Piles. The driven pile capacity was estimated based on the computer software program Driven 1.2 (March 2001), developed by Federal Highway Administration. Due to the variable consistencies of the subsoil layers, local hard driving conditions should be anticipated. However, we recommend that the piles be driven to the Specified tip elevations. The pile capacity will be derived from both frictional resistance along the pile shaft and end bearing. It is anticipated the pile capacity will develop after driving as a result of soil "freeze" and dissipation of excess pore water pressures. The gain of pile capacity after initial driving may be evaluated based on "redriving" after 24-hour (minimum) set-up.

In the event that unanticipated pile driving conditions are encountered, it is recommended that a Pile Driving Analyzer (PDA) be used to evaluate the pile capacity and integrity. Typical applications include capacity evaluation (for both during driving and re-striking) and integrity

testing for piles that have experienced hard driving. The geotechnical engineer should be consulted for any unanticipated pile driving conditions.

Potential Fault Movement/Offset. Note that the existing Nipomo Creek is the trace of the inferred Wilmar Avenue Fault which is considered as potentially active by the County's LHMP. As stated in the USGS Bulletin 1995-AA , the Wilmar Avenue Fault has a slip rate of 0.01 to 0.1 mm/yr. As stated in USGS Bulletin 1995-BB, the post early/late Pliocene vertical slip rates across the Hosgri Fault Zone (which included the Wilmar Avenue Fault in the San Luis/Pismo reach) range from 0.1 to 0.4 mm/yr, but may be as high as 0.44 mm/yr if the rate of right-lateral slip along the fault is greater than 1 mm/yr. In the event there is significant movement during a "single event" the bridge structure could be affected. This should also be recognized and if possible, designed for.

At the time this report is being prepared, no published data is available that predicts the recurrence period along this fault and therefore, no single event offset can be estimated. However, a fault offset should be expected. Based on the above discussion, it is prudent to assume that the fault slip movement will propagate to the surface during design life of the project. The structural design should consider the potential fault slip. The impact of the fault rupture should also be evaluated and incorporated into the structural design and maintenance.

Lateral Design for Piles

For the abutment piles, a pile head deflection of ¼ inch is recommended for vertical piles at service load. Under seismic loading conditions, lateral load analyses using LPILE program were performed for the piles. Plots of deflection, moment, shear and soil reaction are attached in Appendix D. Per recent discussion with Caltrans, group effect for lateral pile resistance analyses was accounted for by adopting a p-y reduction factor of 40% (60% effective) for a pile spacing of 3D.

Lateral Earth Pressures

Bridge abutment should be designed to resist the following applied lateral earth pressures and live loads. These values assume no hydrostatic pore pressure build-up behind the wall and are

based on well-drained backfill behind the walls supported in native soil.

Applied Lateral Earth Pressures

Active Condition	36 pcf (Equivalent Fluid Pressure) for engineered fill
At-Rest Condition	55 pcf (Equivalent Fluid Pressure) for engineered fill
Passive Resistance	5 ksf (ultimate) for seismic design of the abutment wall (5.5 feet high or greater); for less than 5.5 feet high, modify proportionally i.e. 5 x (H/5.5) ksf. A minimum lateral wall movement of 2% of wall height is required to mobilize the full ultimate passive pressure.

Cantilever walls, which are free to rotate at least 0.004 radian, may be assumed flexible for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead, live, or traffic load) should be added to the preceding lateral earth pressures. A coefficient of 0.3 may be used to determine the additional earth pressure resulting from the surcharge.

Embankment Settlements & Waiting Period

Embankment fill is required at the approach embankments of the proposed Nipomo Creek Bridge. The maximum height of the embankment fill is approximately 10 feet. This embankment has a 2(H):1(V) slope on the sides. The embankment width is approximately 75 feet and 95 feet at the west and east embankment, respectively. The boring data indicated that the subsurface soil conditions generally consist of firm to very stiff clay near the surface, underlain by stiff to very stiff clay and medium dense sand. About 10 feet of firm fat clay was encountered in Boring 07-B-13 near the east approach abutment. Settlement is anticipated due to the placement of the embankment fill.

Under the planned 10 feet of embankment, the settlement was estimated to be on the order of 1 to 2 inches. The majority of the settlement will occur during construction as the load is applied. A waiting period of 30 days after embankment construction is recommended for ground adjustment prior to pile driving at the abutments.

Engineered Fill. Engineered fill should be non-expansive and consist of relatively granular material having a P.I. of less than 20. In addition, we recommend that the material within 3 feet of the proposed pavement subgrade have a minimum R-values of 15. The fill shall also be free of organic and inorganic debris, rubles and any other deleterious material.

Compaction of Fill and Subgrades. The recommendations for compaction as per Caltrans standards are as follows:

- 90% for subgrade preparation, general fill and backfilling after removing buried utilities and depressions caused due to construction activities, etc.

- 95% for all engineered fill for structural backfill of bridge abutments, footing subgrade, and for upper 6 inches of pavement subgrade and aggregate base of pavement sections.

Slope Ratio. For permanent fill slope, a maximum slope gradient of 2H:1V with erosion control is recommended. The end slopes at the abutments should not be steeper than 1.5H:1V provided they are protected from the creek flow and surface erosion. Typical slope protection includes slope paving or other approved measures. For temporary slopes during construction, a slope ratio of up to 1H:1V may be used under dry conditions and up to 1.5H:1V under submerged conditions. It should be noted that local irregularities such as loose layers and pockets might require flattening of the slope.

Scour

Scour is the phenomenon when sediments (sand and rocks) are washed away from the bed of a river leaving local depressions and/or holes at riverbed. Although scour may occur at any time, scour action is especially strong during floods. Swiftly flowing water has more energy than calm water to lift and carry sediment down creek. Historic scour was not noticeable along the creek channel. The Hydrology and Hydraulic report prepared by R & M Design Group for the project indicates that the stream will experience aggradation during the design life. The anticipated total scour ranges from 5.3 ft (Abutment 1), 4.8 ft (Main channel at Piers) to 1.5 ft (Abutment 6). The foundation design has accounted for the scour.

Seismic Design Criteria

Based on the seismic hazard map prepared by Mualchin (Caltrans 1996), the governing fault for the proposed structure is the Santa Maria River-Foxen Canyon Fault (SMF, Mw=6.5, style of faulting: Reverse) located within one km from the site. The recommended Peak Bedrock Acceleration for design is 0.7 g.

The recommended curve is based on Caltrans Seismic Design Criteria (Version 1.4). A fault map and the ARS Design Curve are attached with this memo. The criteria are as follows:

1. Soil Profile D
2. ARS Design Curve – Figure B.7 (Caltrans SDC v. 1.4). The base curve is obtained by multiplying the 0.6 g curve data by 1.2. The curve is then modified for directivity by 20% increase of Sa for T > 1 sec., no change of Sa for T < 0.5 sec., and linear interpolation between 0.5 & 1 sec.

Based on the geologic data reviewed, the liquefaction potential is considered moderate.

Corrosion

Per Caltrans guidelines[1] and based on the corrosion test data, the subsurface soil at this site is not considered corrosive. Corrosion data is presented in the following table.

CORROSION TEST RESULTS

Boring	Location		Depth (ft)	Minimum Resistivity (ohms-cm)	pH	Chloride (ppm)	Sulfate (ppm)
	Sta.	Offset (ft)*					
07-NP-1	372+70	80 Lt.	7	880	7.38	20	35.9
07-NP-2	372+12	42 Lt.	6	700	7.38	21.4	19.4
07-B-13	373+30	110 Lt.	5	540	7.92	38.1	120.0

* Ref. "W" Line

[1] "Corrosion Guidelines Version 1.0", California Department of Transportation, Division of Engineering Services, Material Engineering and Testing Services, Corrosion Technology Branch, , September 2003

Plan Review

We recommend that final plans for foundations be reviewed by this office prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

Construction Observation

To a degree, the performance of any structure is dependent upon construction procedures and quality. Hence, observation of grading, foundation excavations, pile installations should be carried out by the regulating agencies. If the subsurface conditions different from those forming the basis of our recommendations are encountered, this office should be informed in order to assess the need for design changes. Therefore, the recommendations presented in this report are contingent upon good quality control and these geotechnical observations during construction.

INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices and are based on our site reconnaissance and the assumption that the subsurface conditions do not deviate from observed conditions. All work done is in accordance with generally accepted geotechnical engineering principles and practices. No warranty, expressed or implied, of merchantability or fitness, is made or intended in connection with our work or by the furnishing of oral or written reports or findings. The scope of our services did not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in structures, soil, surface water, groundwater or air, below or around this site. Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require that additional expenditures be made during construction to attain a properly constructed project. Some contingency fund is thus recommended to accommodate these possible extra costs.

This report has been prepared for the proposed structure as described earlier, to assist the

engineer in the design of this project. In the event any changes in the design or location of the facilities are planned, or if any variations or undesirable conditions are encountered during construction, our conclusions and recommendations shall not be considered valid unless the changes or variations are reviewed and our recommendations modified or approved by us in writing.

This report is issued with the understanding that it is the designer's responsibility to ensure that the information and recommendations contained herein are incorporated into the project and that necessary steps are also taken to see that the recommendations are carried out in the field.

The findings in this report are valid as of the present date. However, changes in the subsurface conditions can occur with the passage of time, whether they be due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or from the broadening of knowledge. Accordingly, the findings in this report might be invalidated, wholly or partially, by changes outside of our control.

Respectfully submitted,
PARIKH CONSULTANTS, INC.

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Senior Engineer

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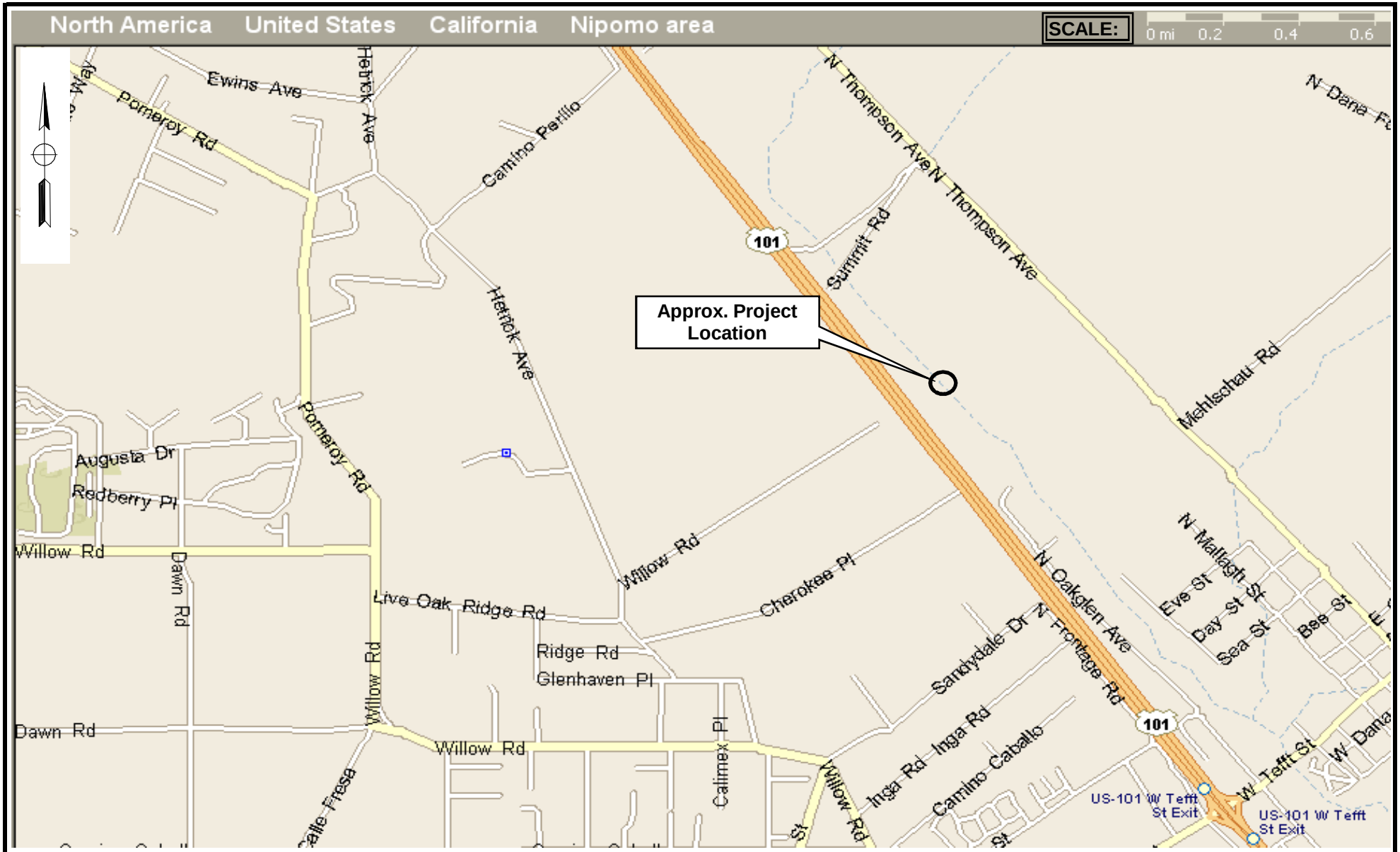
Project Manager

James B. Baker, C.E.G. 1021
Engineering Geologist



SOURCE:
Microsoft Streets & Trips 2002

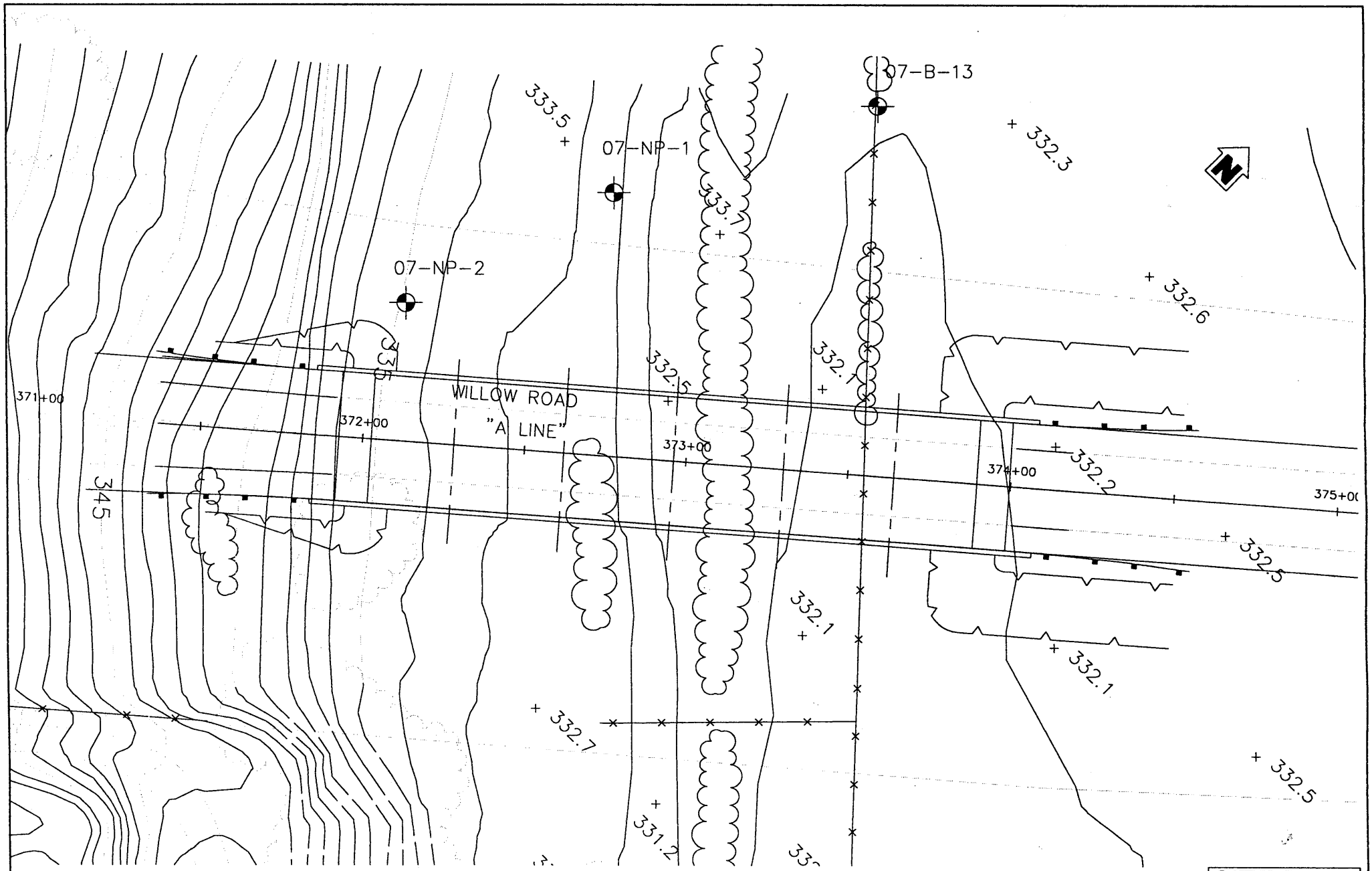
PROJECT LOCATION MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

NIPOMO CREEK BRIDGE
COUNTY OF SAN LUIS OBISPO, CALIFORNIA

JOB NO.: 203125.NPO

PLATE NO.: 1



Note: All units are in feet unless otherwise specified
Reference Map was provided by Rajappan & Meyer

SCALE 1 inch = 40 feet

SITE PLAN

LEGEND

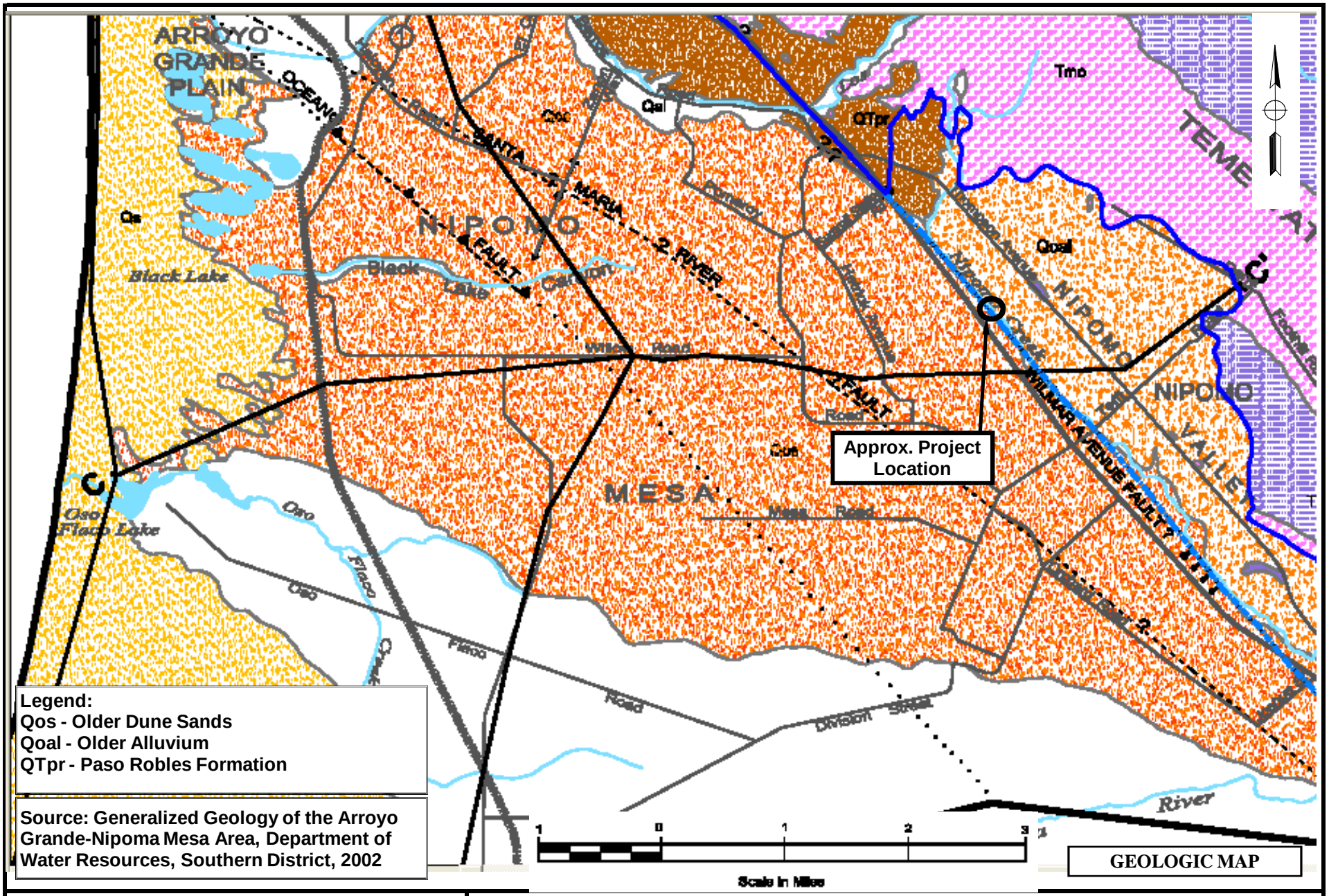
07-NP-1
Approx. Soil Boring Location

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

NIPOMO CREEK BRIDGE
COUNTY OF SAN LUIS OBISPO, CALIFORNIA

JOB NO.:203125.NPO

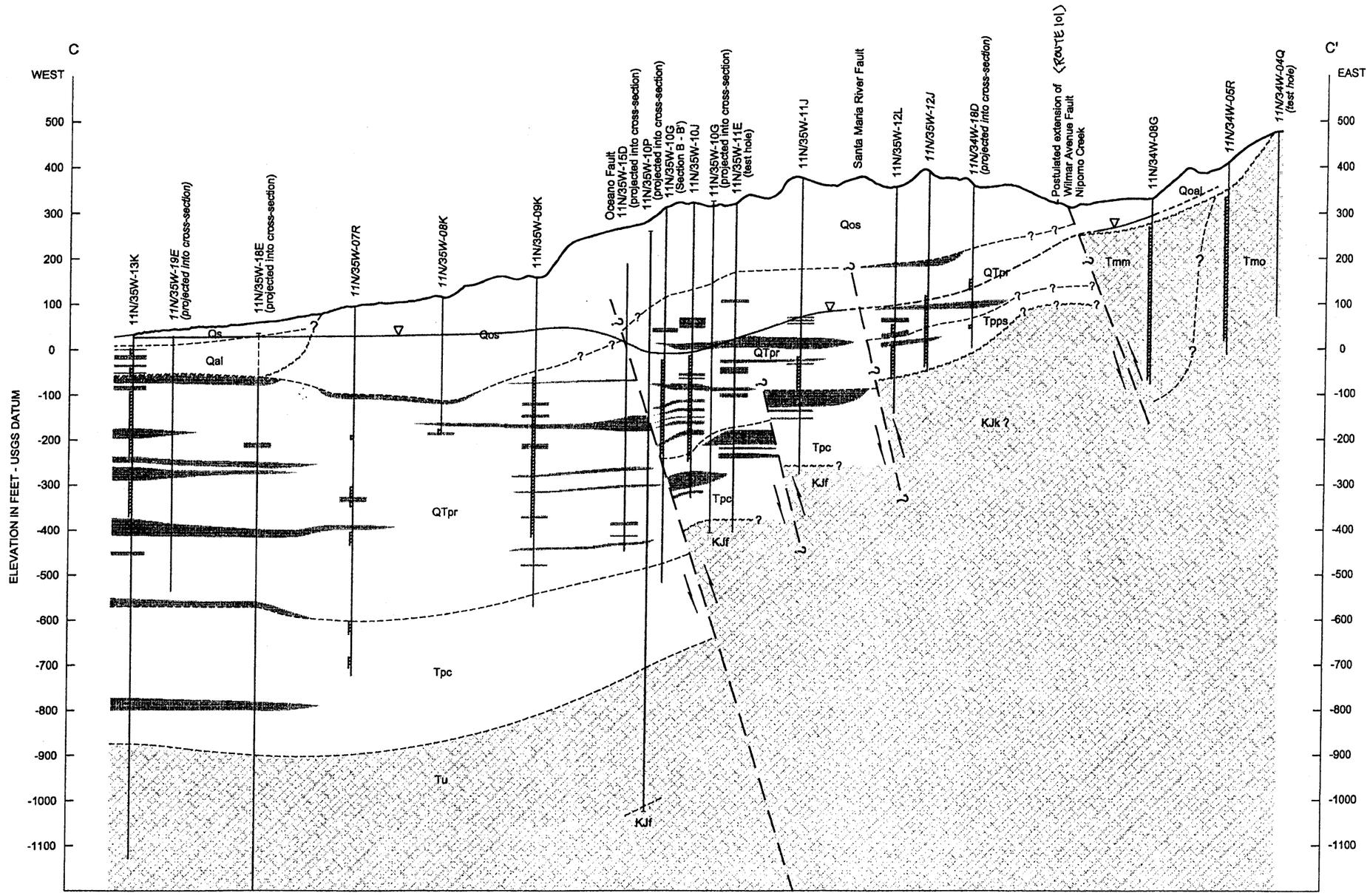
PLATE NO. 2



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

NIPOMA CREEK BRIDGE
COUNTY OF SAN LUIS OBISPO, CALIFORNIA

JOB NO.: 203125.NPO	PLATE NO.: 3A



EXPLANATION

Formation and Symbol	
Qs	Dune Sand
Qal	Alluvial Deposits
Qos	Older Dune Sand
Qoal	Older Alluvial Deposits
QTpr	Paso Robles Formation
Tpc	Careaga Formation
Tpps	Squire Member, Pismo Formation
Tmm	Monterey Formation
Tmo	Obispo Formation
Tu	Undifferentiated Tertiary Deposits
KJk	Knoxville Formation
KJf	Franciscan Complex

12N/36W-36L	Water well with electric and geologic log
32S/12E-13L	Water well with geologic log
10N/36W-02G	Oil well with electric log, dashed above record
11N/35W-10P	Oil well with geologic log, dashed above record

Hydrogeologic Unit

- Unconsolidated to weakly cemented clayey silt to silty clay with minor sand and gravel
- Unconsolidated to weakly cemented sand, gravel, silt, sandstone with minor clay and silty clay
- Bedrock

- Perforated interval
- Fault, arrows indicate relative vertical movement
- Unconformity
- Contact
- 1995 Groundwater Elevation

Note: References include: Department of Water Resources, 1970; Buchanan-Banks et al., 1978; Hall, 1981; Nitchman, 1988; Hanson et al., 1994; Lettis et al., 1994.

Location of geologic section shown on Plate 2

Vertical scale 20 times horizontal scale

Horizontal scale in feet

4000 0 4000

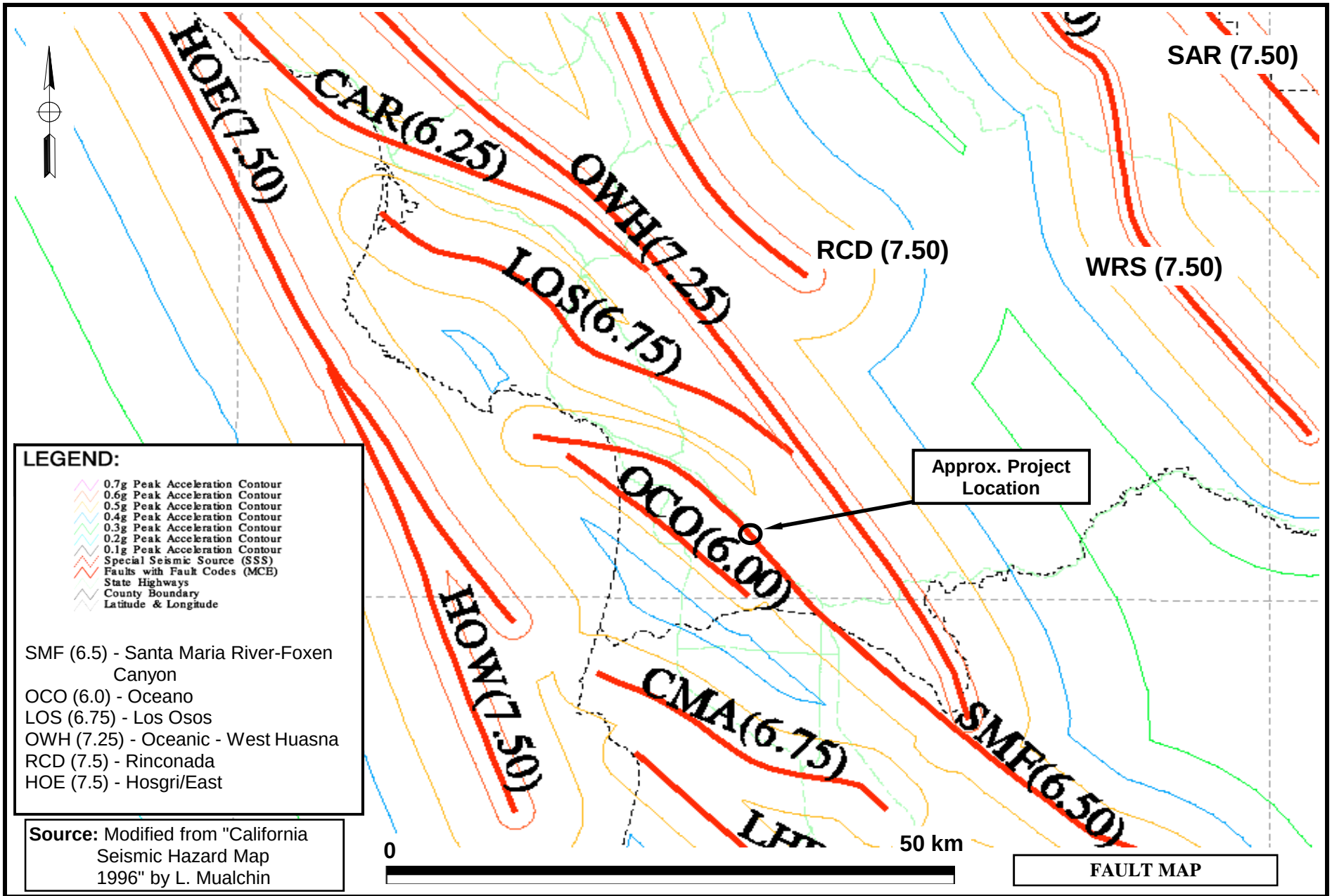
WATER RESOURCES OF THE
ARROYO GRANDE - NIPOMO MESA AREA

◇◇◇

PLATE NO.: 3-B

GEOLOGIC SECTION C - C'

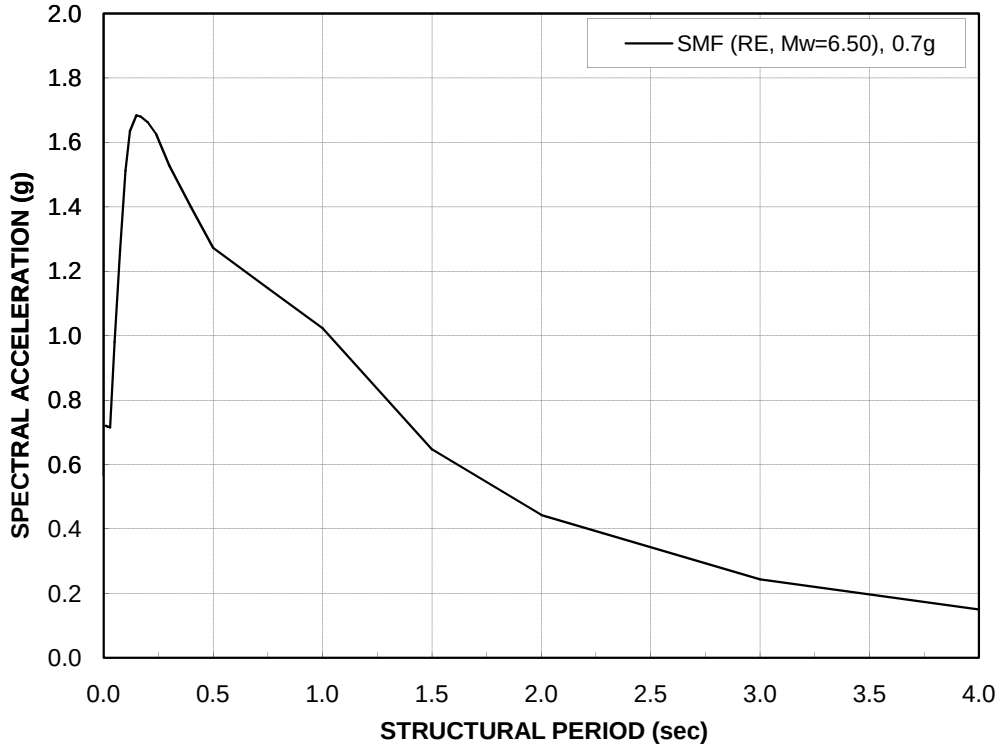
Department of Water Resources, Southern District, 2002



FAULT MAP

PARIKH CONSULTANTS, INC. **GEOTECHNICAL CONSULTANTS** **MATERIALS TESTING**	**NIPOMO CREEK BRIDGE** **COUNTY OF SAN LUIS OBISPO, CALIFORNIA**
	JOB NO.: 203125.NPO — **PLATE NO.: 4**

ARS DESIGN CURVE
NIPOMO CREEK BRIDGE
SAN LUIS OBISPO COUNTY, CALIFORNIA



Spectral Data

Period (sec)	Spectral Accel. (g)
0.010	0.720
0.020	0.717
0.030	0.716
0.050	0.980
0.075	1.263
0.100	1.512
0.120	1.635
0.150	1.684
0.170	1.680
0.200	1.663
0.240	1.626
0.300	1.528
0.400	1.399
0.500	1.272
0.750	1.148
1.000	1.024
1.500	0.647
2.000	0.443
3.000	0.243
4.000	0.150

Caltrans SDC (v 1.4), Figure B.7,
Governing Fault: Santa Maria River - Foxen Canyon Fault
(Mw = 6.5, Soil Profile Type D, PBA = 0.7 g)
The base curve is obtained by multiplying the 0.6g curve data by 1.2.
The curve is then modified for directivity by 20% increase of Sa for T > 1sec., no change of Sa for T < 0.5 sec. and linear interpolation between 0.5 and 1 sec.

APPENDIX A

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
05	SLO	RTE 101	–		

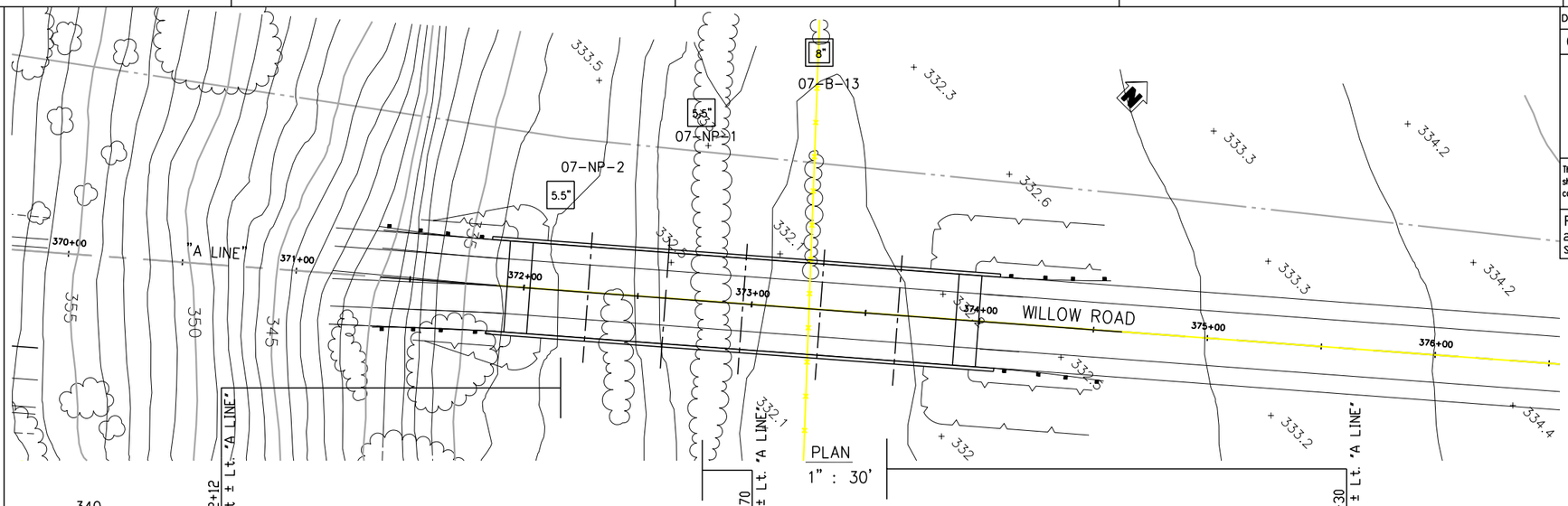
GEOTECHNICAL PROFESSIONAL DATE

PLANS APPROVAL DATE

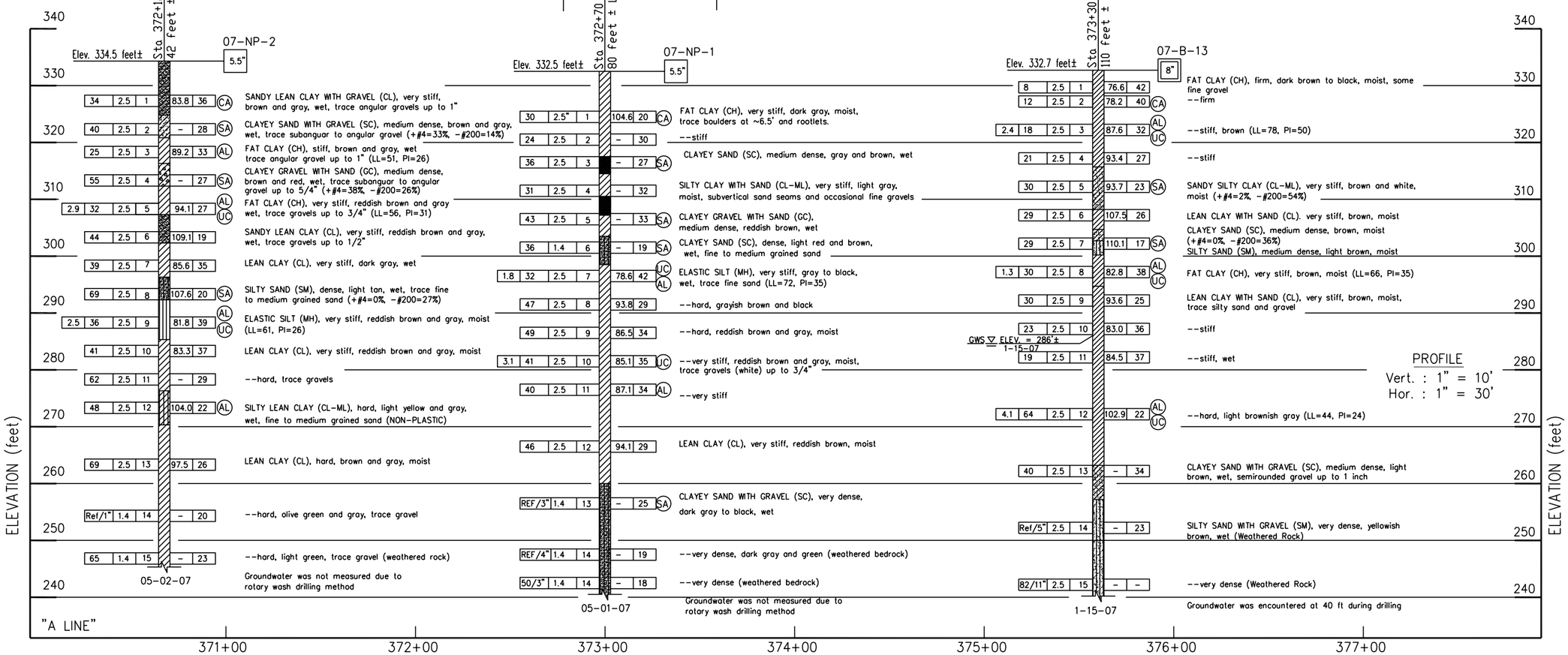
The County of San Luis Obispo or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

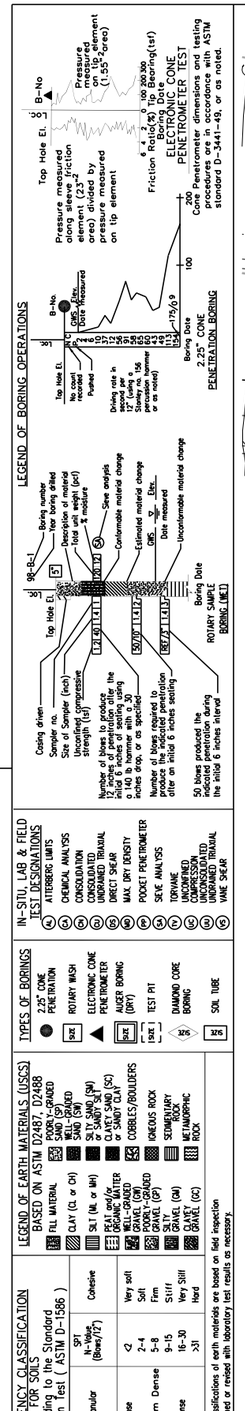
REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/09 — GEOTECHNICAL — STATE OF CALIFORNIA

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE CA 95131



Note:
Standard Penetration Test Sampler: I.D. = 1.4"; O.D. = 2" Modified California Sampler: I.D. = 2.5"; O.D. = 3" Hammer Assembly: A 140 lb hammer with a 30" drop





X DESIGN OVERSIGHT	DRAWN BY	LAM TRAN	PERRY CHAN — FIELD INVESTIGATION BY:
X SIGN OFF DATE	CHECKED BY	AMIT BHARADWAJ	DATE: MAY 2007

PREPARED FOR THE COUNTY OF SAN LUIS OBISPO DEPARTMENT OF PUBLIC WORKS

DAVID WANG — PROJECT ENGINEER

BRIDGE NO. –
POST MILE –

NIPOMO CREEK BRIDGE
LOG OF TEST BORINGS

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU
EA

DISREGARD PRINTS BEARING EARLIER REVISION DATES 06/29/09

REVISION DATES (PRELIMINARY STAGE ONLY) — SHEET — OF

FILE => $REQUEST

USERNAME => $USER — DATE PLOTTED => $DATE — TIME PLOTTED => $TIME

OSF GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 2/25/05)

PLATE A-1

APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

The field classification of the samples was visually verified in the laboratory according to the Unified Soil Classification System. The results are presented in the Log of Test Borings, Appendix A.

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM Test Method D 2216-92. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the "Log of Borings", Appendix A.

Atterberg Limits

The Atterberg Limits were determined for selected samples of the fine-grained materials. These results were used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM Test Method D 4318-93. The results of these tests are presented on Plate B-2, “Plasticity Chart”.

Grain Size Classification

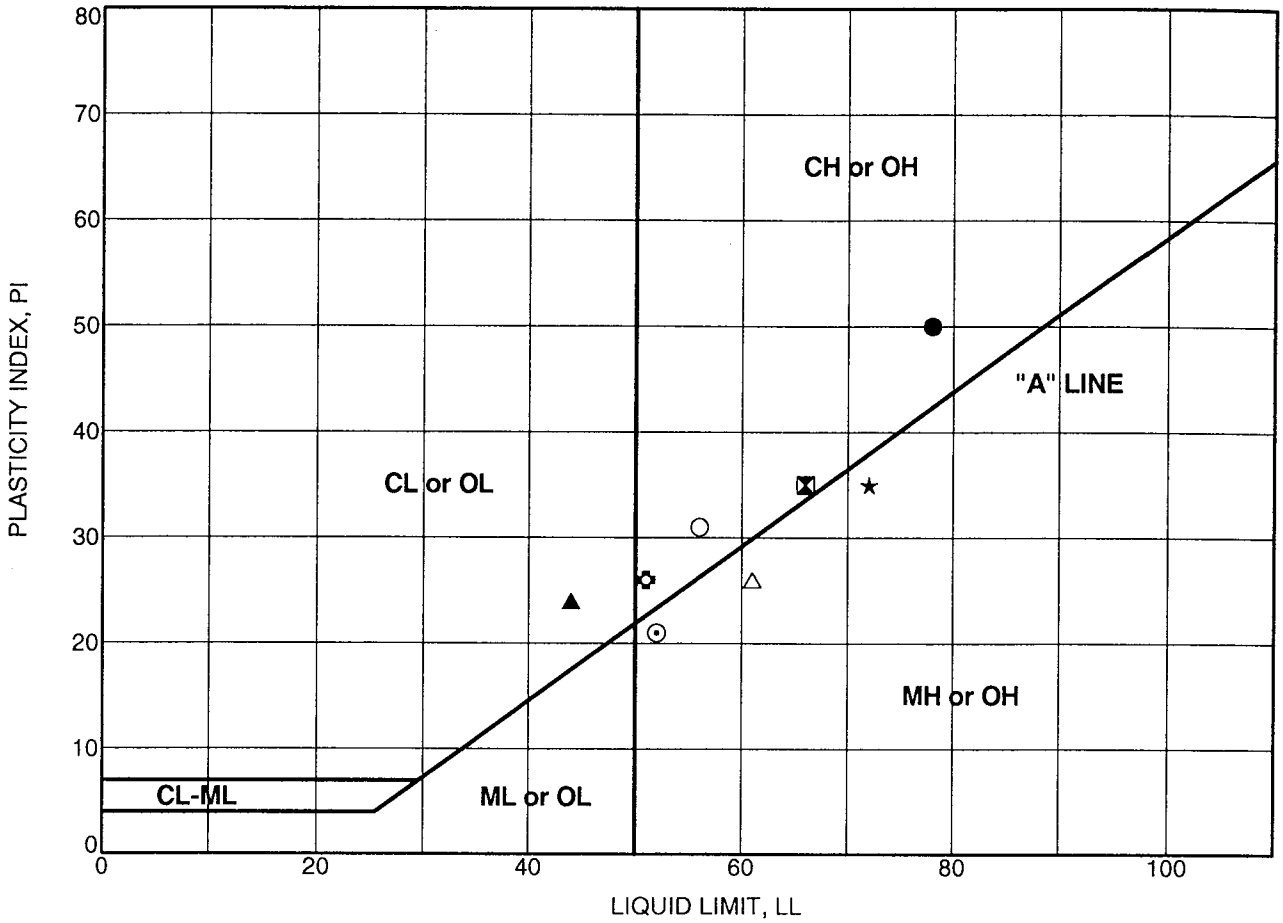
Grain size classification tests (ASTM Test Method D 422-63) were performed on selected samples of granular soil to aid in the classification. The results are presented on Plate B-3, "Grain Size Distribution Curves".

Unconfined Compression Tests

Strength tests were performed on selected undisturbed samples using unconfined compression machine. Unconfined compression tests were performed in general accordance with ASTM Test Method D 2166-91. The results are presented on “Log of Test Borings”, Appendix A.

Corrosion Tests

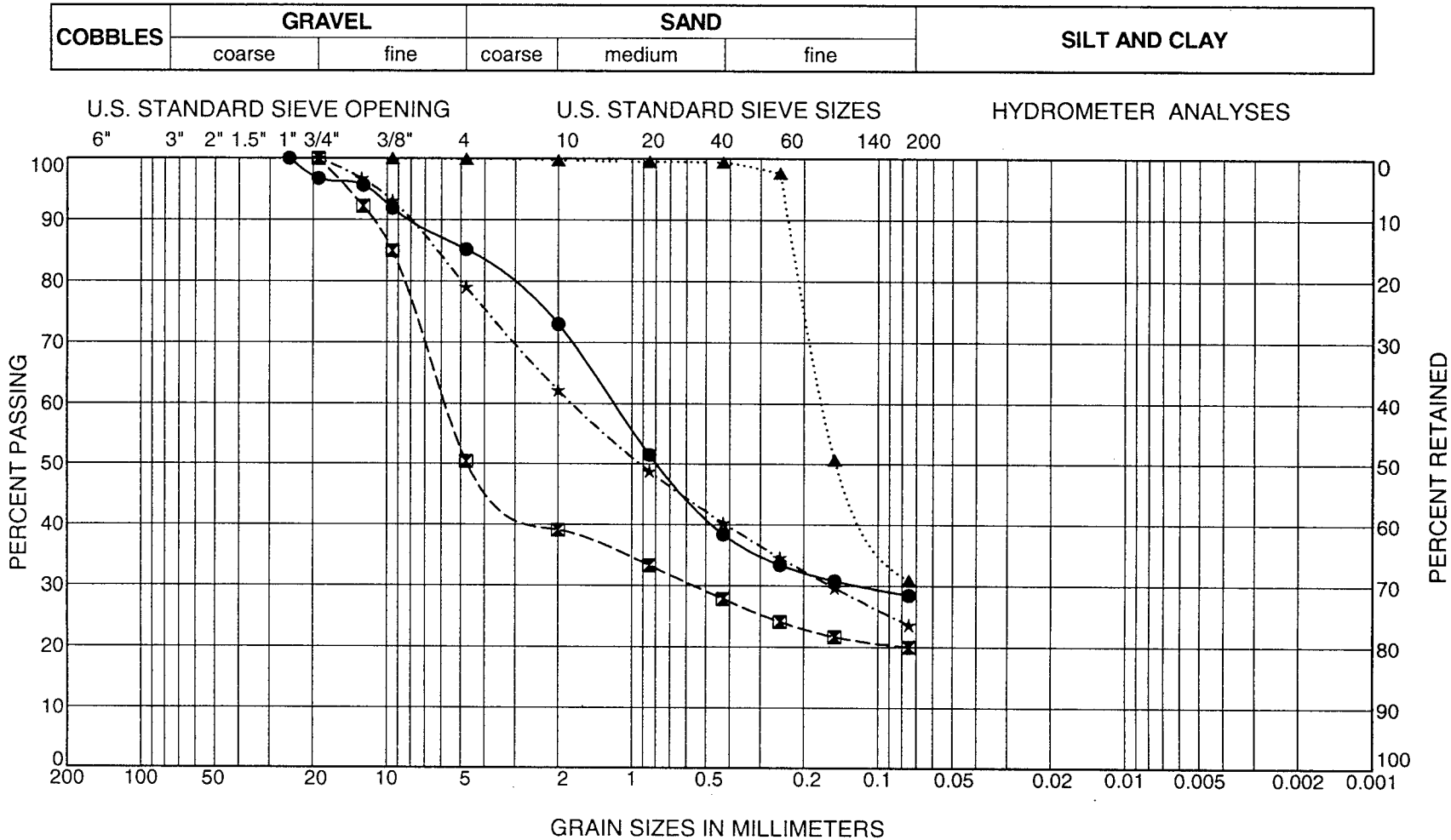
Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistively tests were performed according to California Test Method 643. Sulfate and chloride tests were performed by AnaCon Testing Laboratory. The test results for water-soluble sulfate and chloride contents are presented on Plates B-4.



PLASTICITY CHART

Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
B-13	3	9.5	●		78	28	50	FAT CLAY (CH)
B-13	8	34.5	⊠		66	31	35	FAT CLAY (CH)
B-13	12	59.5	▲		44	20	24	LEAN CLAY (CL)
NP-1	7	35.0	★		72	37	35	ELASTIC SILT (MH)
NP-1	11	55.0	⊙		52	31	21	ELASTIC SILT (MH)
NP-2	3	15.0	✿		51	25	26	FAT CLAY (CH)
NP-2	5	25.0	○		56	25	31	FAT CLAY (CH)
NP-2	9	45.0	△		61	35	26	ELASTIC SILT (MH)

GRAIN SIZE DISTRIBUTION CURVES



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
NP-1	3	15.0	●			CLAYEY SAND (SC)
NP-1	5	25.0	⊠			CLAYEY GRAVEL WITH SAND (GC)
NP-1	6	30.0	▲			CLAYEY SAND (SC)
NP-1	13	75.0	★			CLAYEY SAND WITH GRAVEL (SC)

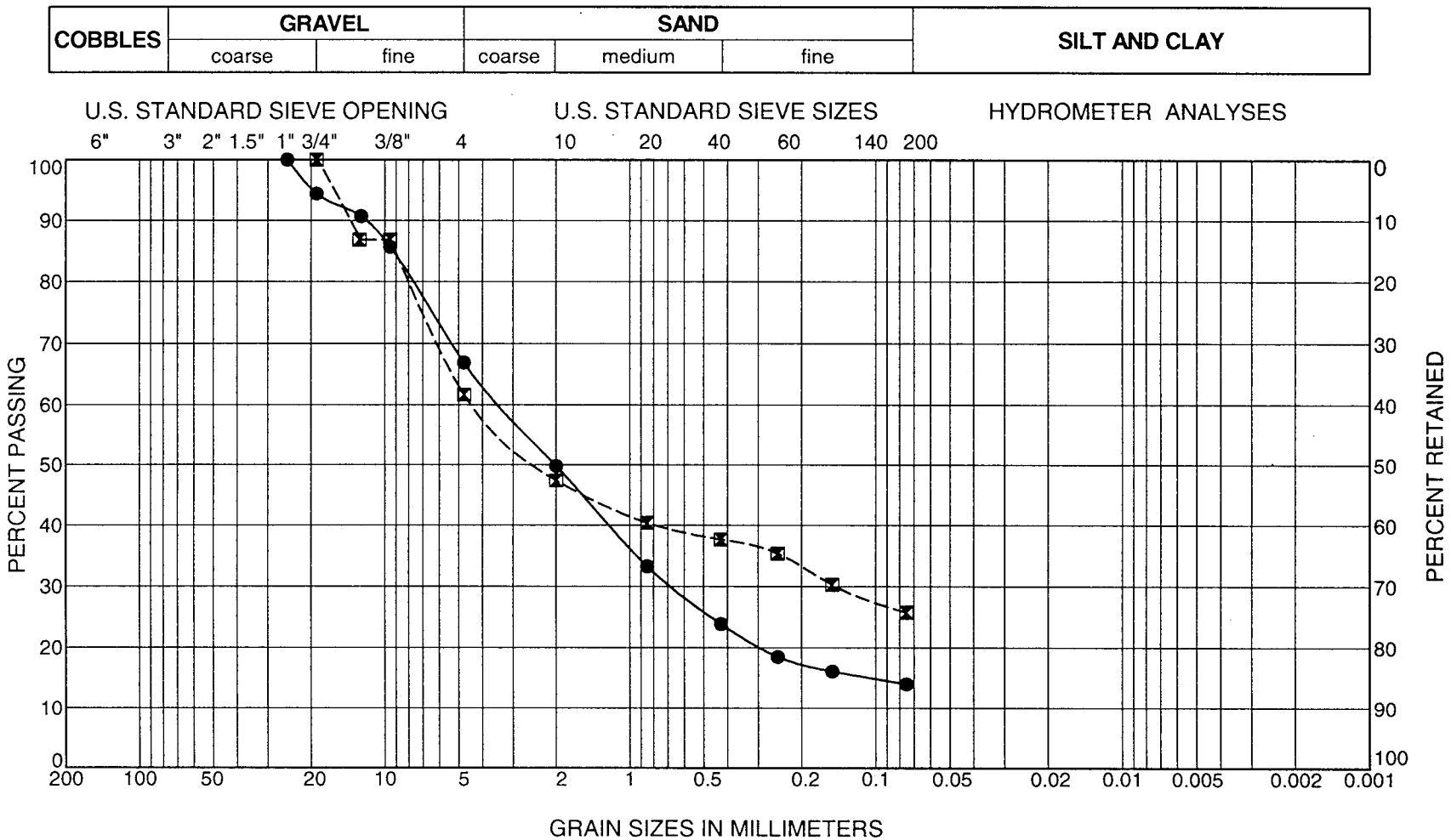
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

NIPOMO CREEK BRIDGE AT WILLOW ROAD
COUNTY OF SAN LUIS OBISPO, CA

JOB NO: 203125.NPO

PLATE NO: B-3A

GRAIN SIZE DISTRIBUTION CURVES



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
NP-2	2	11.0	●			CLAYEY SAND WITH GRAVEL (SC)
NP-2	4	20.0	⊠			CLAYEY GRAVEL WITH SAND (GC)

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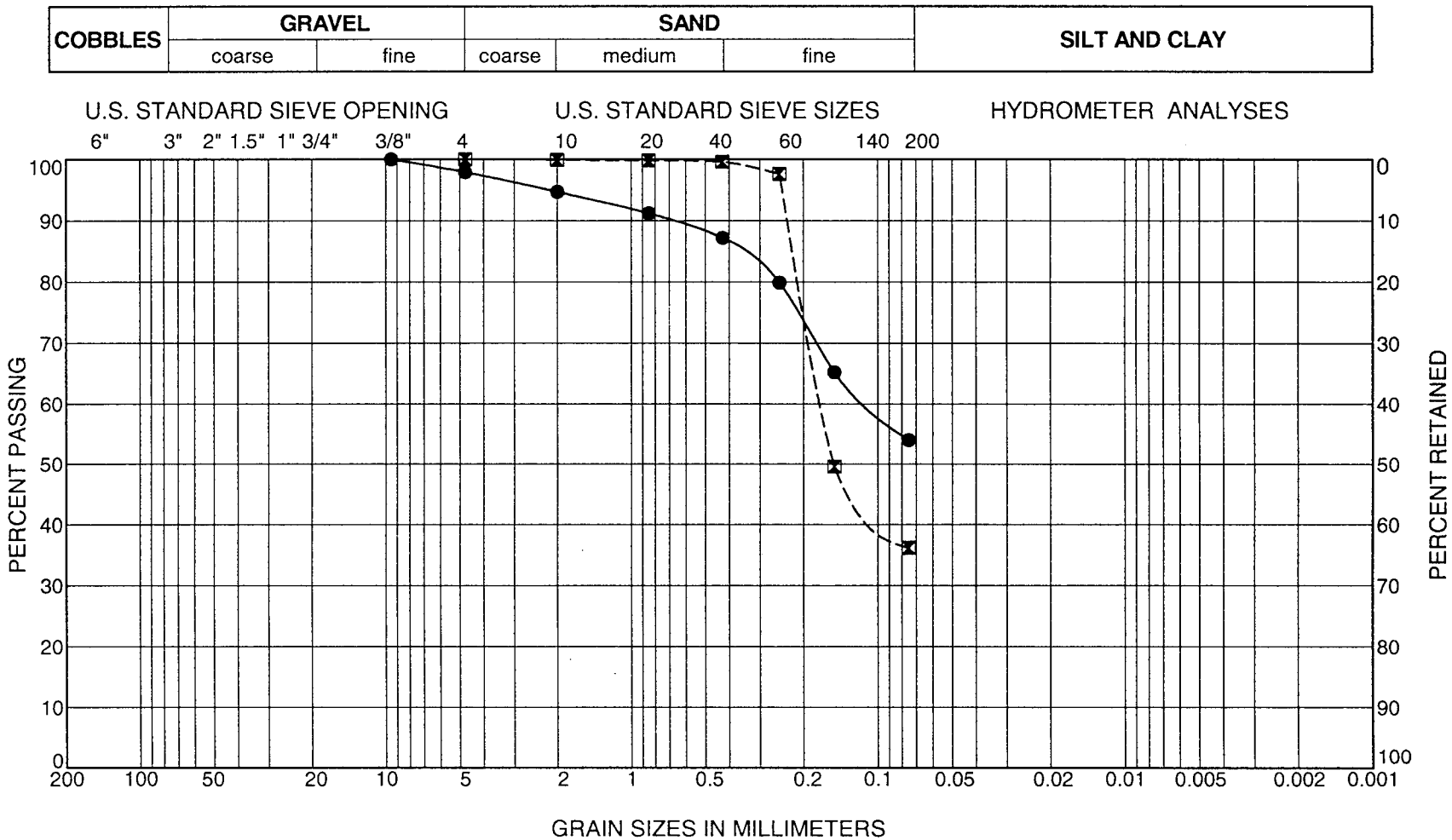
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

NIPOMO CREEK BRIDGE AT WILLOW ROAD
COUNTY OF SAN LUIS OBISPO, CA

JOB NO: 203125.NPO

PLATE NO: B-3B

GRAIN SIZE DISTRIBUTION CURVES



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
B-13	5	19.5	●			SANDY SILTY CLAY (CL-ML)
B-13	7	29.5	⊠			CLAYEY SAND (SC)

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

NIPOMO CREEK BRIDGE AT WILLOW ROAD
COUNTY OF SAN LUIS OBISPO, CA

JOB NO: 203125.NPO

PLATE NO: B-3C

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 05/23/2007
Date Submitted 05/18/2007

To: Prav Dayah
Parikh Consultants, Inc.
356 S. Milpitas Blvd.
Milpitas, Ca 95035

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 203125 NPO WILLOW RD Site ID : NP-1 #1@7'.
Thank you for your business.

* For future reference to this analysis please use SUN # 50573-100953.

EVALUATION FOR SOIL CORROSION

Soil pH	7.38	
Minimum Resistivity	0.88 ohm-cm (x1000)	
Chloride	20.0 ppm	00.00200 %
Sulfate	35.9 ppm	00.00359 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

PLATE NO: B-4A

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 05/23/2007
Date Submitted 05/18/2007

To: Prav Dayah
Parikh Consultants, Inc.
356 S. Milpitas Blvd.
Milpitas, Ca 95035

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager RH

The reported analysis was requested for the following location:
Location : 203125 NPO WILLOW RD Site ID : NP-2 #1@6'.
Thank you for your business.

* For future reference to this analysis please use SUN # 50573-100954.

EVALUATION FOR SOIL CORROSION

Soil pH	7.38	
Minimum Resistivity	0.70 ohm-cm (x1000)	
Chloride	21.4 ppm	00.00214 %
Sulfate	19.4 ppm	00.00194 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

PLATE NO: B-4B

APPENDIX C

NORCAL GEOPHYSICAL CONSULTANTS, INC.

March 27, 2007

Mr. Parry Chan
Civil Engineer
Parikh Geotechnical Consultants, Inc.
356 S. Milpitas Blvd.
Milpitas, CA 95035

Subject: NIPOMO CREEK BRIDGE
GEOPHYSICAL SURVEY
ELECTRICAL RESISTIVITY PROFILING
NIPOMO, CA
NORCAL JOB NUMBER 07-426.08

Attention: Mr. Chan

Gentlemen:

This report presents the findings of a surface geophysical survey performed by NORCAL Geophysical Consultants for the subject project. The survey was part of an on-going geotechnical site evaluation being performed by Parikh Consultants, Inc. for the construction of an interchange and road extension from US Route 101 north of the city Nipomo, California. The field survey was conducted on March 5, 2007 by NORCAL geophysicist William J. Henrich and geophysical technician Ken McGuire.

1.0 SITE DESCRIPTION

The proposed interchange and road extension project is located approximately one mile northwest of the city of Nipomo (see Plate 1, Location Map). Geologically, the site is situated on a series of gently sloping coalescing alluvial fans that originate from the southwest flank of the Sierra Madre Mountains. The alluvial fans are underlain by geologically recent unconsolidated alluvial deposits of clay, silt, sand and gravel. Bedrock may consist of sandstone belonging to the Obispo Formation. The site is drained by the northwest -southeast trending Nipomo Creek. On the northwest side of Nipomo Creek, the topography becomes hilly and was noted in the field to consist of very sandy alluvium possibly sand dune deposits.

321A BLODGETT STREET • COTATI, CA 94931 • TELEPHONE (707) 796-7170 • FAX (707) 796-7175
www.norcalgeophysical.com

2.0 PURPOSE

In the vicinity of Nipomo, the potentially active Wilmar Avenue Fault generally trends parallel to US Route 101. This fault may transect the site that is less than 500 feet northeast of US Route 101. The road extension project calls for a bridge to span across Nipomo Creek. Therefore, the surface electrical resistivity profiling was conducted to investigate for possible subsurface lateral lithologic or structural variations that may be associated with recent or older faulting at this bridge site.

3.0 SCOPE OF WORK

Our scope of work consisted of acquiring two electrical resistivity profiles oriented perpendicular to the trend of Nipomo Creek and to the extent constrained by vegetation, flanking the bridge site. These profiles will be analyzed and interpreted as resistivity models of the subsurface and described and interpreted in a written report.

4.0 METHODOLOGY

The electrical resistivity of the subsurface is measured using a galvanic resistivity method. This consists of transmitting electrical current into the earth through pairs of grounded metal electrodes, and measuring the resulting potential drop across the second pair of grounded metal electrodes. There are a variety of electrode arrangements (arrays) that can be used. The dipole-dipole electrode configuration was used in this survey because it provides information on both the depth and lateral extent of subsurface electrical properties.

5.0 DATA ACQUISITION

Apparent resistivity data was acquired with an automatic multi-electrode system manufactured by Advanced Geosciences Incorporated (AGI, Austin, Texas). The AGI system consists of an electrode switching box, acquisition console and four interconnecting cables capable of addressing 56 stainless steel electrodes. The 56 electrodes were laid out in a co-linear array and planted in the ground with the distance between each electrode set at a constant spacing. Both resistivity profiles contained a electrode spacing of 10 feet.

Data acquisition is controlled by programmed commands whereby one electrode pair (dipole) is energized by electric current and an adjacent electrode pair measures the resulting voltage drop. This measurement cycle takes 2 seconds. The instrument advances to the next cycle but moves the voltage measuring dipole down the array by turning the appropriate electrode switches on and off. Subsequent readings are then taken by incrementally increasing the distance between dipoles up to six times the initial electrode separation. More readings are taken by systematically increasing dipole lengths which are taken out to another six times the dipole separation out to the end of the array. All total, 691 electrical resistivity measurements were collected below a 56 electrode array.

We collected two resistivity profiles labeled NIPO-1 and 2. The locations of these electrical resistivity profiles is shown on Plate 1. The profile alignment (southwest - northeast) was chosen to intercept the trends of suspected geologic contacts and lateral discontinuities potentially associated with faulting.

6.0 DATA ANALYSIS

Upon completion of a dipole-dipole survey, apparent resistivity data are downloaded from the AGI console to a lap-top computer using the computer program STINGDMP. The data are inverted to true resistivity versus depth and distance models using the computer modeling program EARTH-IMAGER 2D. Both programs were written by AGI. We have incorporated the true terrain elevations into the resistivity models. The data generated by the EARTH-IMAGER 2D program are then gridded and contoured using the computer program Surfer 8.0 by Golden Software to produce final 2-D resistivity models.
A critical reduction procedure which we used in our presentation of the 2-D resistivity models was that we optimized the color scale within each resistivity model to enhance subsurface features. The result was that fine features possibly related to alluvial bedding and folding could be compared on both models.

7.0 PRESENTATION AND INTERPRETATION

The results of the resistivity modeling are shown as subsurface electrical resistivity profiles on Plate 2. In general, analysis of the electrical resistivity ER profiles showed that the local subsurface resistivity distribution ranged from 2 to over 1000 ohm-meters. However, we reduced the resistivity range to 2 to 100 ohm-m (see Resistivity Scale below each resistivity profile) to enhance resistivity variations of the alluvium which typically lie in this range. These resistivities are represented as a contour map which was in filled with a color spectrum. The purple-blue shades represent the lowest values of the local resistivity spectrum; the red-yellow shades represent the highest resistivity values. Based on the distribution of subsurface resistivity, inferences as to the configuration of geologic layers or bodies can be made and depending on the magnitude of resistivity and other subsurface information, a general interpretation of the underlying lithology.

In general, ER Profiles NIPO-1 and 2 show complex distributions of resistivity highs (red) on the southwest and lows (blues) on the northeast. Based on resistivity magnitudes and considering geologic information reported in Boring B-13 (see NIPO-1, Station 345) we have annotated the profiles with a very basic geologic interpretation. We have subdivided the geologic units into dune, alluvial clay, silt, sand and gravel deposits and bedrock. The subsurface configuration of these geologic units is shown on each profile.

In comparing the two profiles, a major, common subsurface feature appears as a near vertical lateral resistivity change approximately 50 feet west of the existing stream channel. This vertical boundary appears to separate primarily high resistivity zone interpreted as bedrock on the southwest from low resistivity zone interpreted as alluvium on the northeast given the limited geologic information available. This discontinuity can be explained by the presence of an interpreted near vertical fault boundary. This fault trend is northwest coincident with the course of Nipomo Creek and parallel to US Route 101.

Our results show a subsurface structure that is highly suggestive of a fault. However, even such a very generalized depiction of the subsurface will require a subsequent program of trenching and drilling to validate our geophysical interpretation.

8.0 LIMITATIONS

The scope of NORCAL's services for this project consisted of using geophysical techniques to measure and map the extent of subsurface physical properties. The accuracy of our findings is subject to specific site conditions and limitations inherent to the techniques used. We performed our services in a manner consistent with the level of skill ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

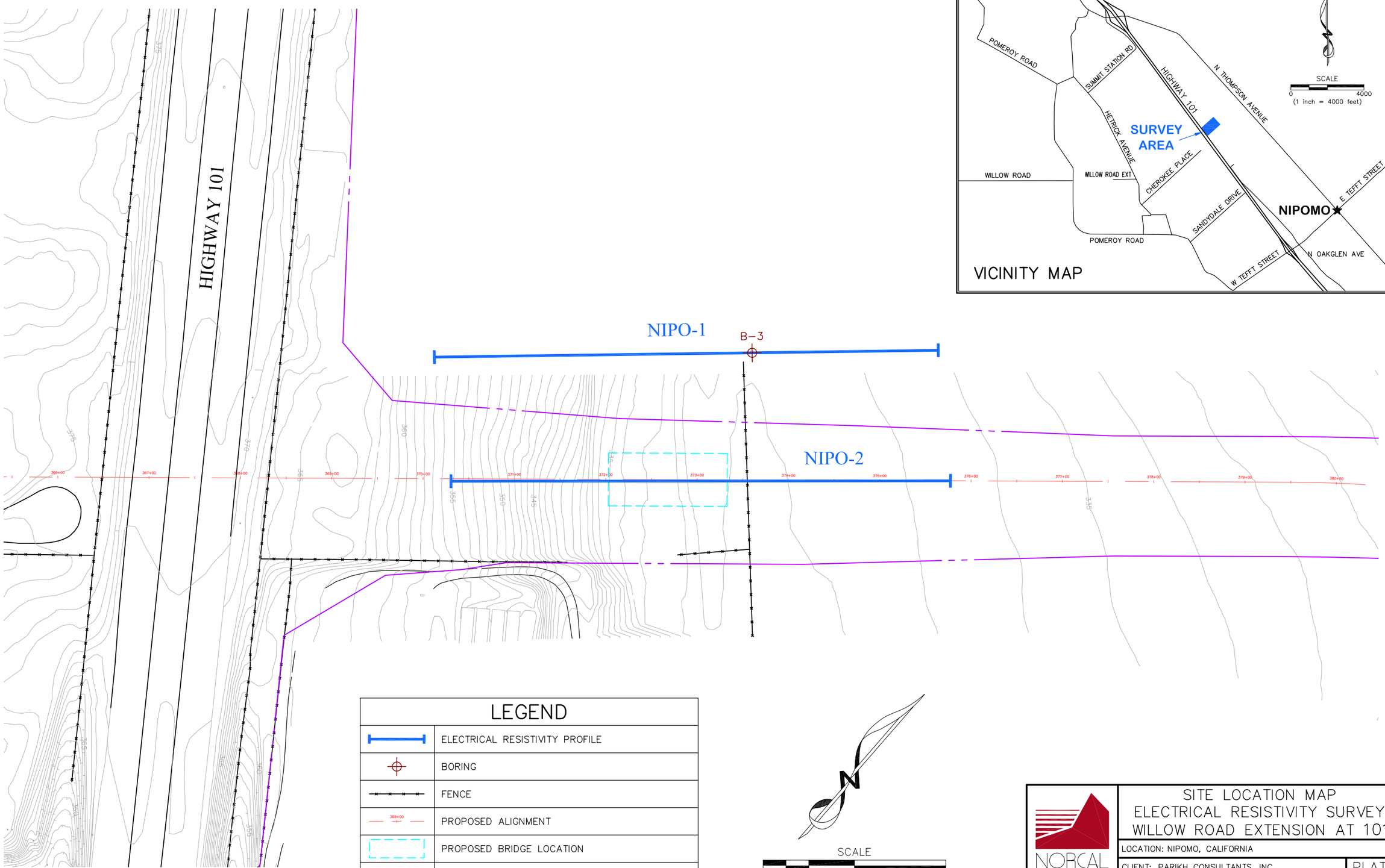
We appreciate the opportunity to provide our services to Parikh Geotechnical Consultants, Inc. for this project. If you have any questions, or require additional geophysical services, please do not hesitate to call.

Yours very truly,

NORCAL Geophysical Consultants, Inc.

William J. Henrich
Geophysicist PG-893

Enclosures: Plates 1 and 2



NORCAL

SITE LOCATION MAP
ELECTRICAL RESISTIVITY SURVEY
WILLOW ROAD EXTENSION AT 101

LOCATION: NIPOMO, CALIFORNIA

CLIENT: PARIKH CONSULTANTS, INC.

JOB #: 07-426.08

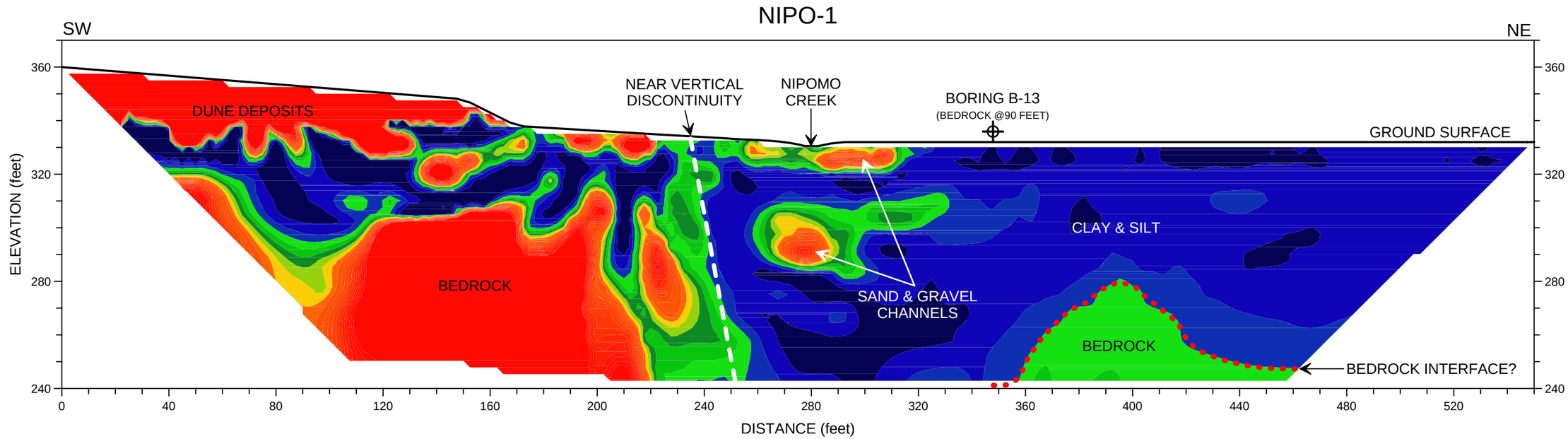
NORCAL GEOPHYSICAL CONSULTANTS INC.

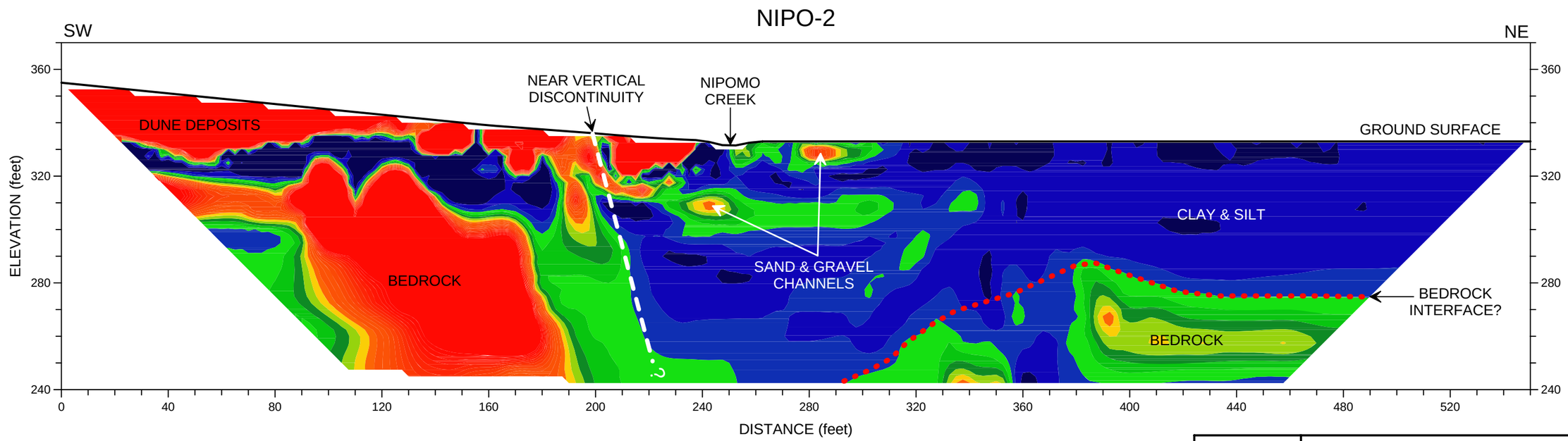
DATE: MAR. 2007

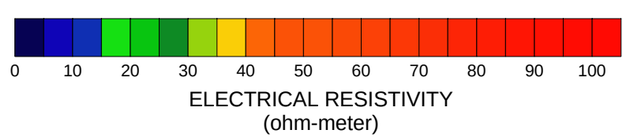
DRAWN BY: G.RANDALL

APPROVED BY: WJH

PLATE 1







SCALE: 1 INCH = 40 FEET

NORCAL

ELECTRICAL RESITIVITY PROFILES
NIPO-1 & NIPO-2
WILLOW ROAD EXTENTIONS AT 101

LOCATION: NIPOMO, CALIFORNIA

CLIENT: PARIKH CONSULTANTS, INC.

JOB #: 07-426.08

NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: MAR. 2007

DRAWN BY: G.RANDALL

APPROVED BY: WJH

PLATE 2

APPENDIX D

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: S:\ONGOIN~1\2003\203125~1.NPO\DRIVEN~1\DRIVEN\ABUTEM~1\ABT.DVN
Project Name: Nippomo Creek Bridge Project Date: 06/29/2009
Project Client: R _M
Computed By: Amit
Project Manager: David Wang

PILE INFORMATION

Pile Type: Concrete Pile
Top of Pile: 0.00 ft
Length of Square Side: 14.00 in

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	70.00 ft
	- Driving/Restrike	70.00 ft
	- Ultimate:	70.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

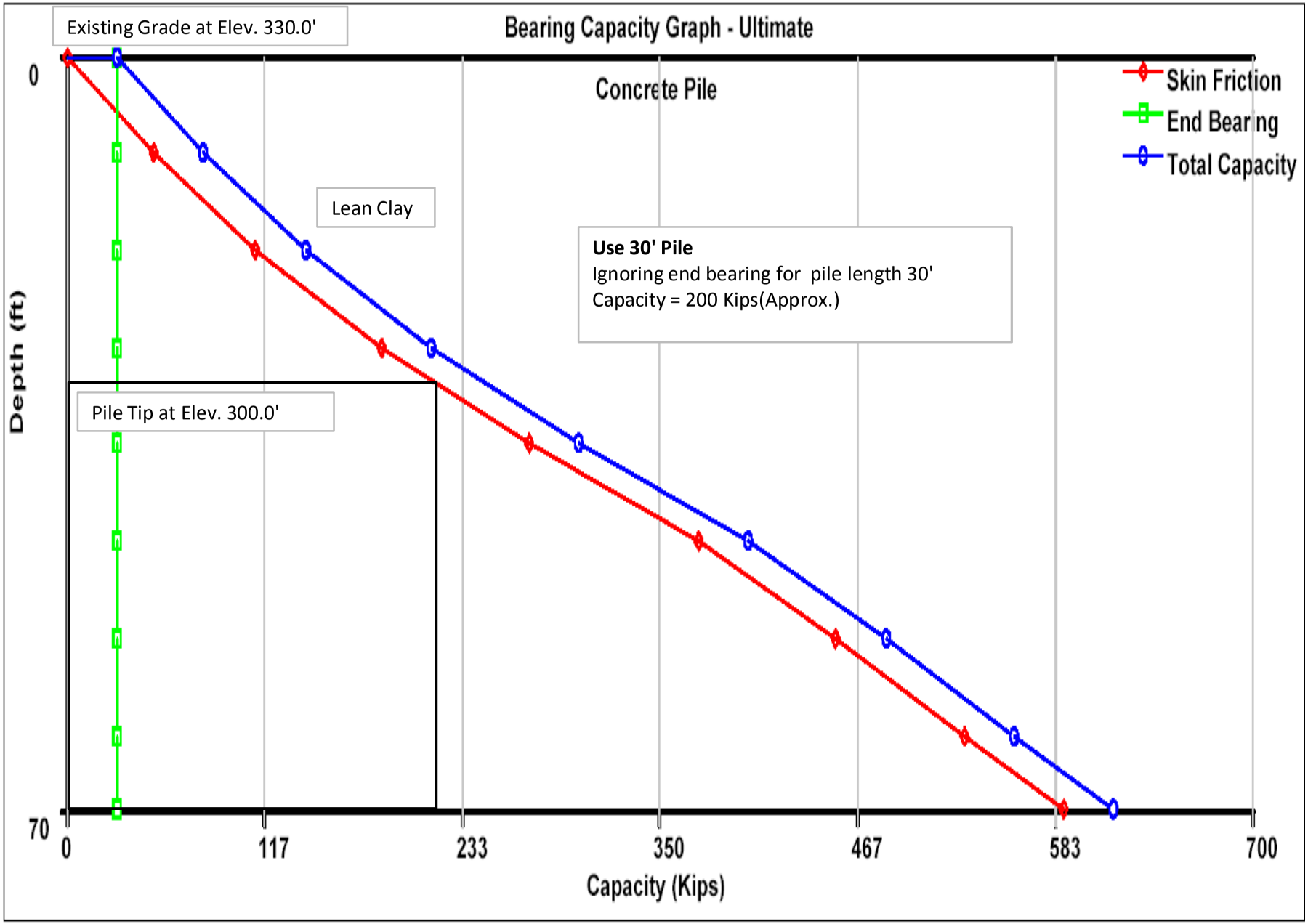
ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	70.00 ft	0.00%	120.00 pcf	2400.00 psf	T-79 Concrete

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.06 Kips	29.40 Kips	29.46 Kips
9.01 ft	51.30 Kips	29.40 Kips	80.70 Kips
18.01 ft	111.37 Kips	29.40 Kips	140.77 Kips
27.01 ft	185.83 Kips	29.40 Kips	215.23 Kips
36.01 ft	272.81 Kips	29.40 Kips	302.21 Kips
45.01 ft	372.32 Kips	29.40 Kips	401.72 Kips
54.01 ft	453.68 Kips	29.40 Kips	483.08 Kips
63.01 ft	529.28 Kips	29.40 Kips	558.68 Kips
69.99 ft	587.92 Kips	29.40 Kips	617.32 Kips

Pile Capacity Evaluation per FHWA "DRIVEN" Program - Nipomo Creek Bridge-Abutment



DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: S:\ONGOIN~1\2003\203125~1.NPO\DRIVEN~1\DRIVEN\PIER\PER.DVN
Project Name: Nippomo Creek Bridge Project Date: 06/29/2009
Project Client: R _M
Computed By: Amit
Project Manager: David Wang

PILE INFORMATION

Pile Type: Concrete Pile
Top of Pile: 0.00 ft
Length of Square Side: 14.00 in

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	70.00 ft
	- Driving/Restrike	70.00 ft
	- Ultimate:	70.00 ft
Ultimate Considerations:	- Local Scour:	12.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

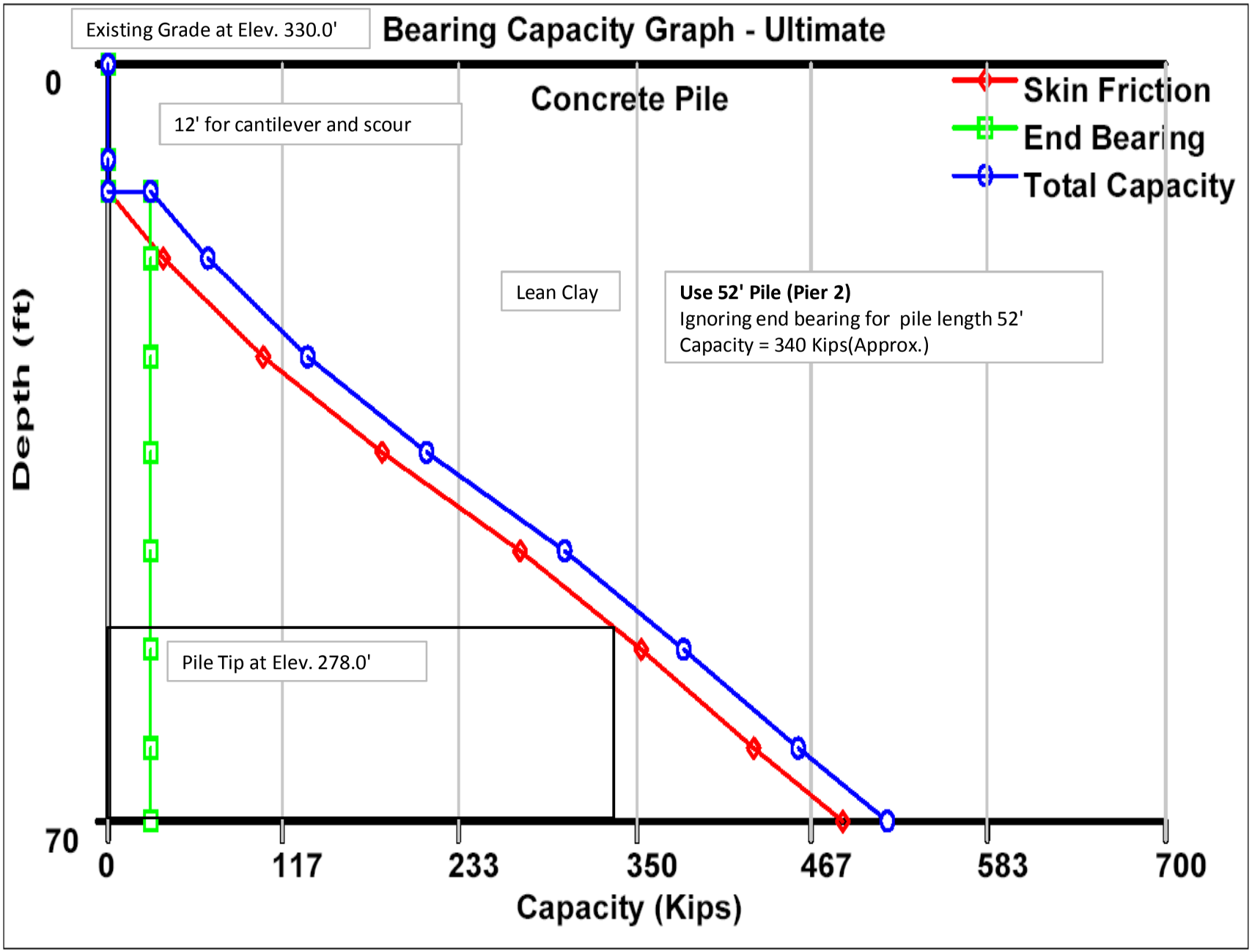
ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	70.00 ft	0.00%	120.00 pcf	2400.00 psf	T-79 Concrete

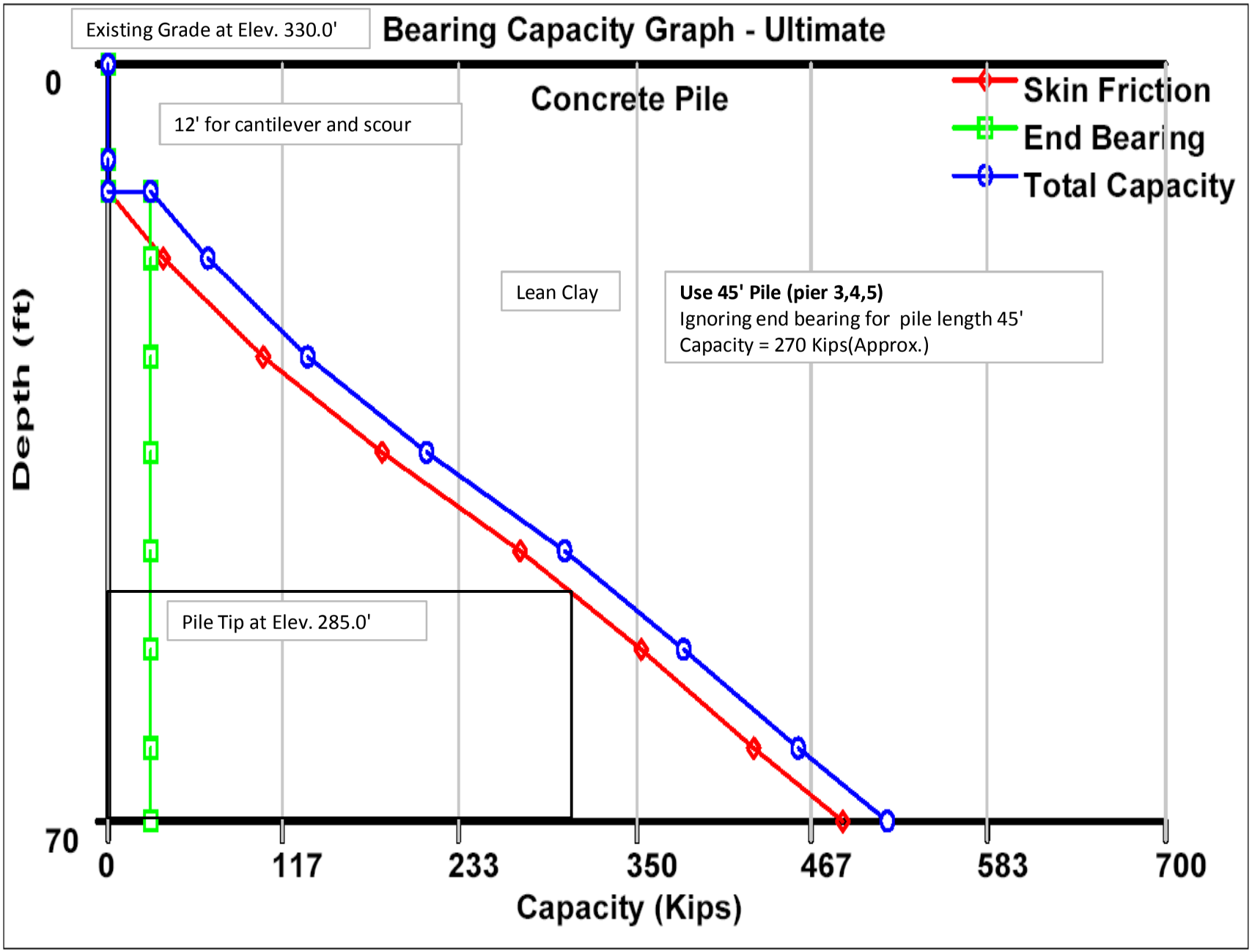
ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
11.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
12.00 ft	0.00 Kips	29.40 Kips	29.40 Kips
18.01 ft	37.17 Kips	29.40 Kips	66.57 Kips
27.01 ft	103.27 Kips	29.40 Kips	132.67 Kips
36.01 ft	181.90 Kips	29.40 Kips	211.30 Kips
45.01 ft	273.05 Kips	29.40 Kips	302.45 Kips
54.01 ft	352.88 Kips	29.40 Kips	382.28 Kips
63.01 ft	428.48 Kips	29.40 Kips	457.88 Kips
69.99 ft	487.12 Kips	29.40 Kips	516.52 Kips

Pile Capacity Evaluation per FHWA "DRIVEN" Program - Nipomo Creek Bridge-Pier

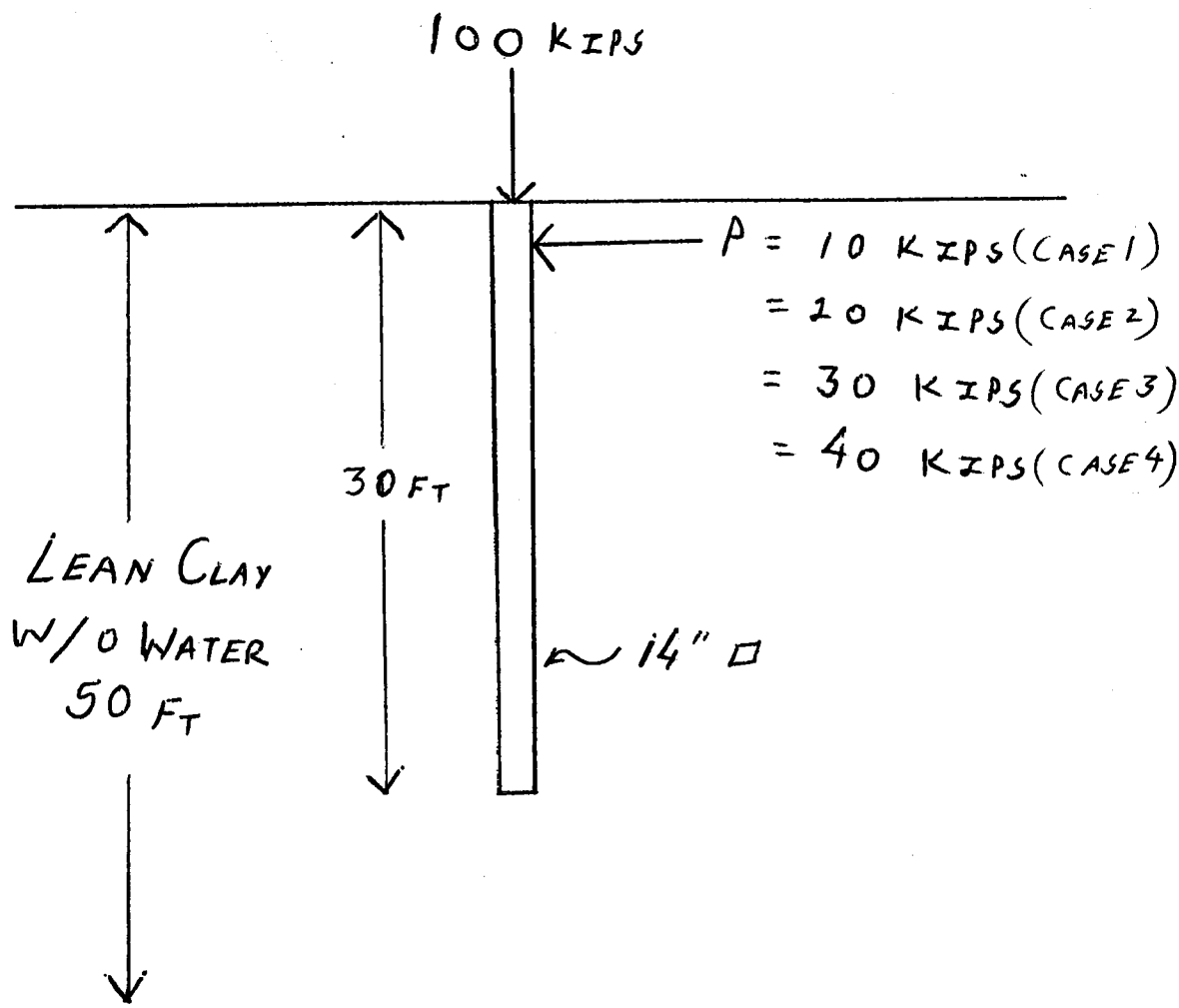


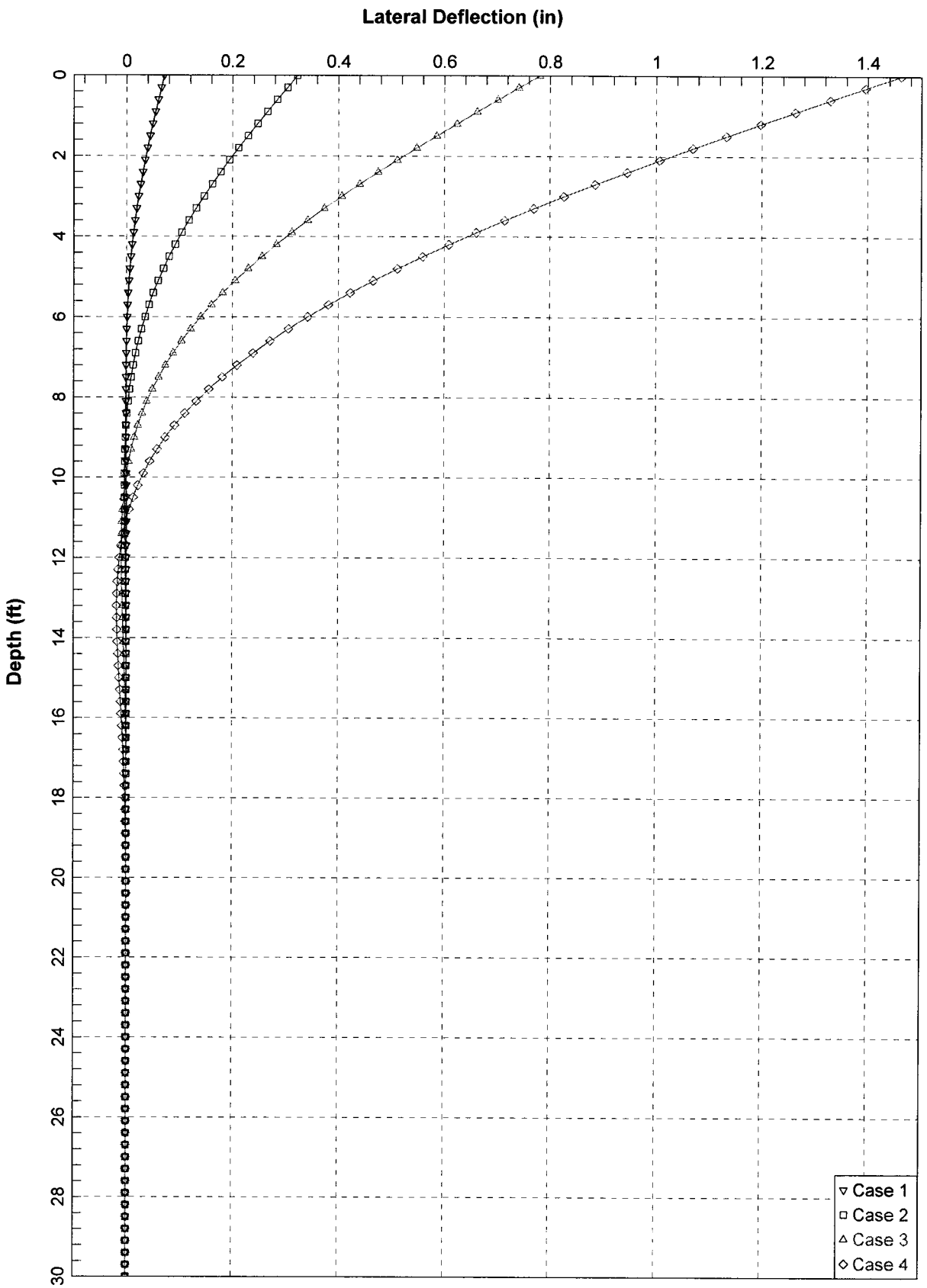
Pile Capacity Evaluation per FHWA "DRIVEN" Program - Nipomo Creek Bridge-Pier

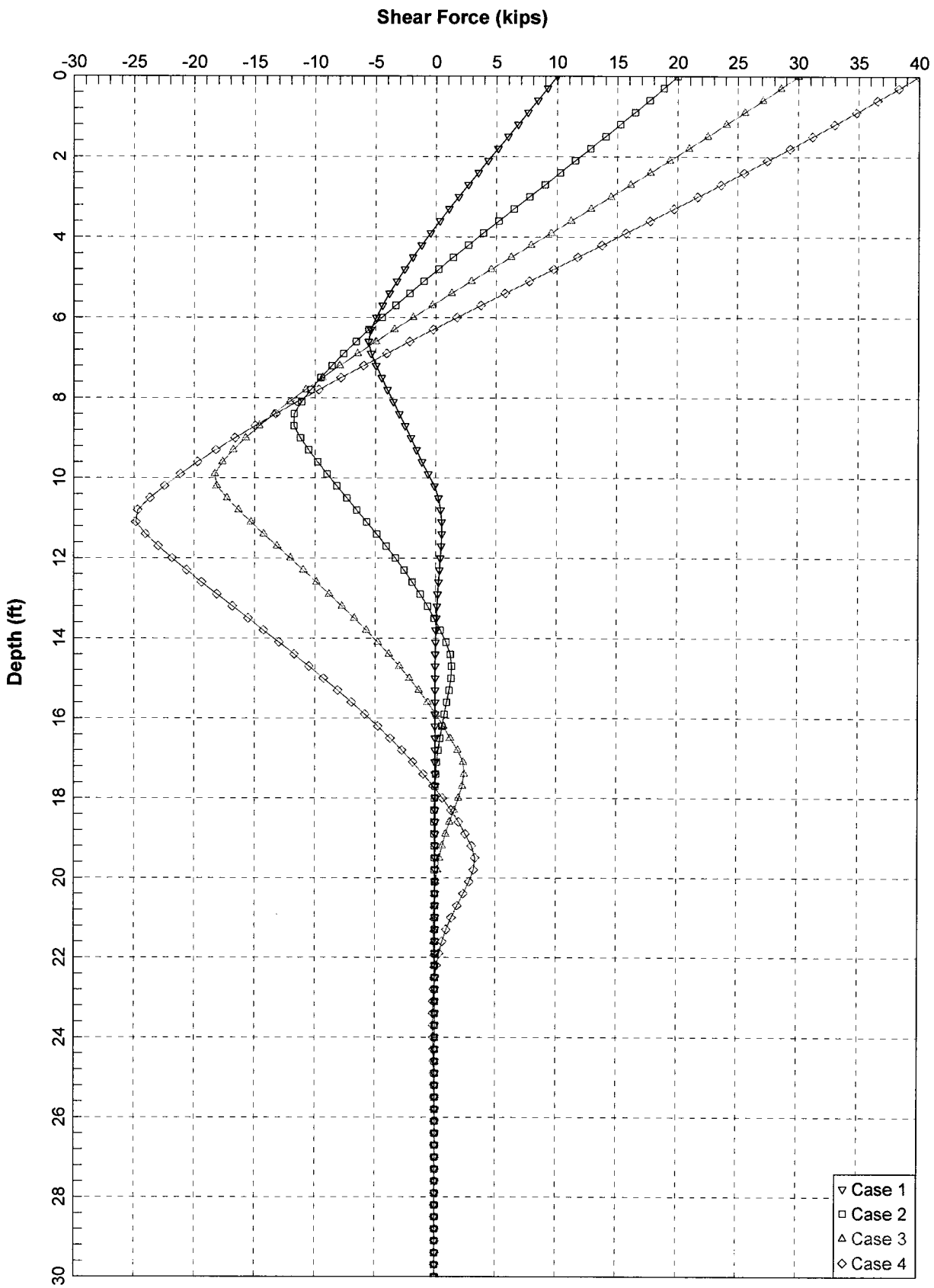


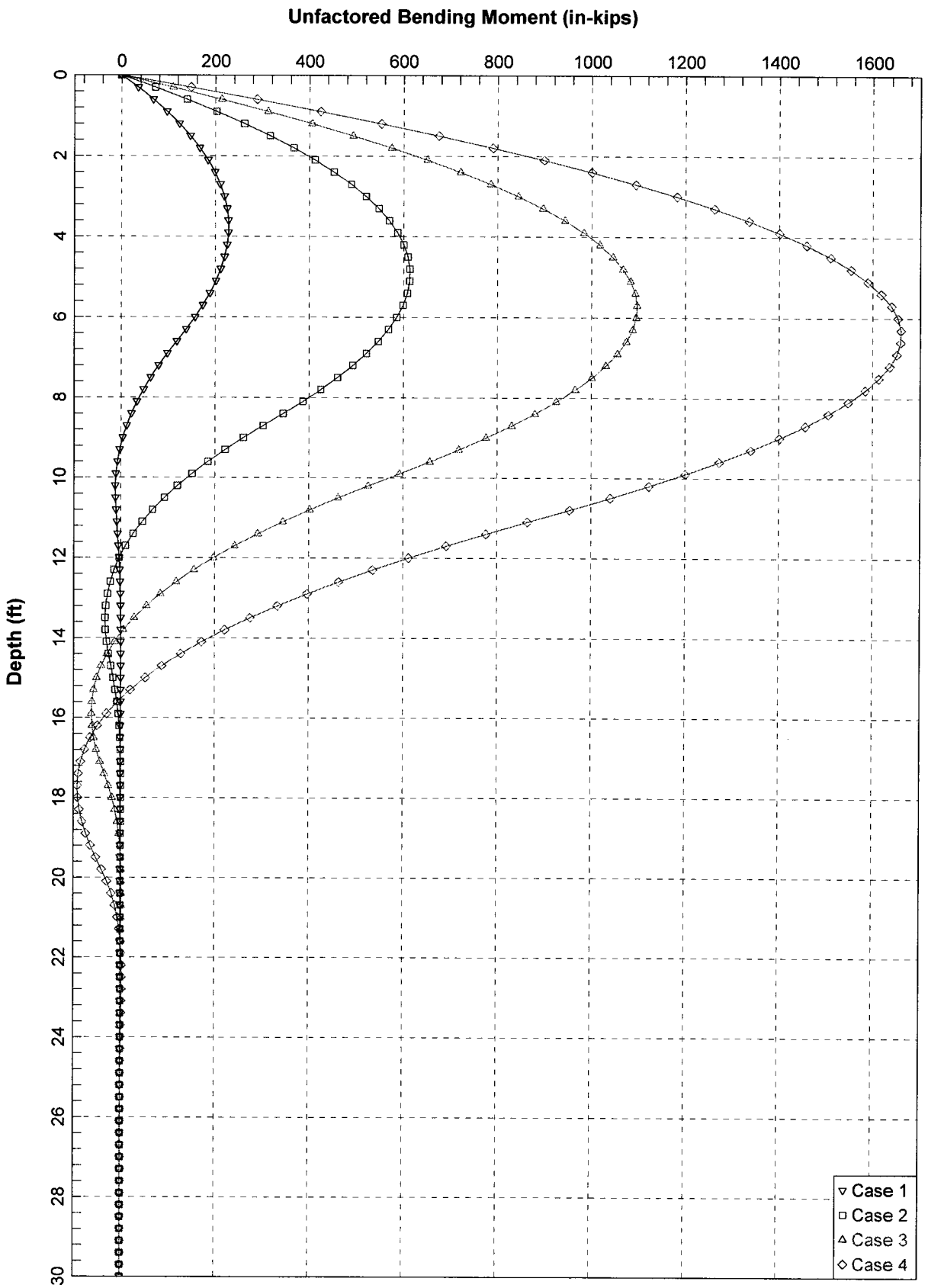
Abutment-100.lpd

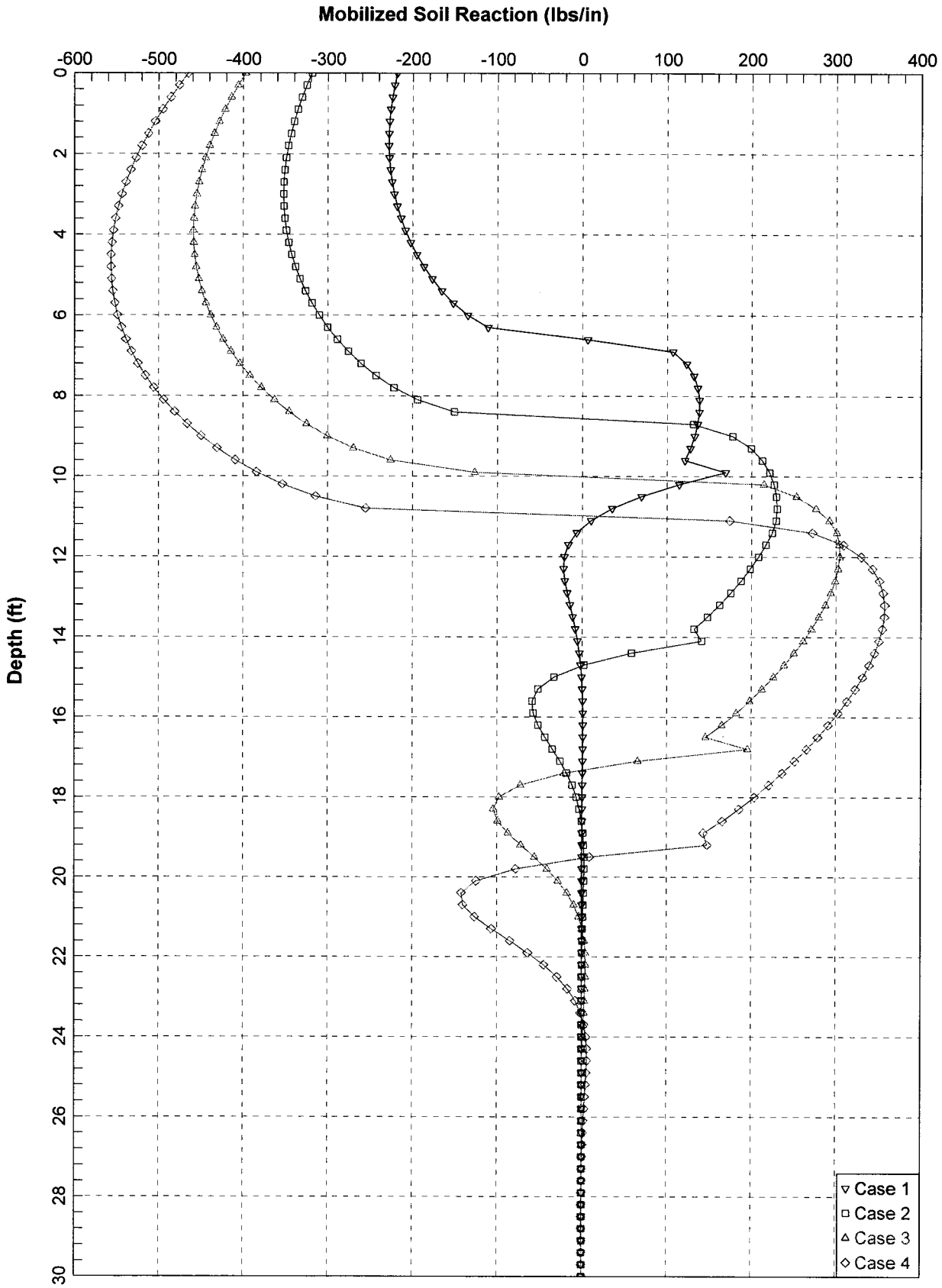
```
LPILEP5
New LPILE Plus 5.0 Data File
1       1       0       0       0       0
100     2       -24     360     0
0       14      3201.33 196     3000000
360     14      3201.33 196     3000000
1       2       2       0       2
3       -24     624     0       0
-24     0.034
624     0.034
-24     16.67   0       0.005   0
624     16.67   0       0.005   0
0       0.6     1
480     0.6     1
0       1       1
4
1       10000   0       100000
1       20000   0       100000
1       30000   0       100000
1       40000   0       100000
0
1       1       0
100     1E-5    100
```



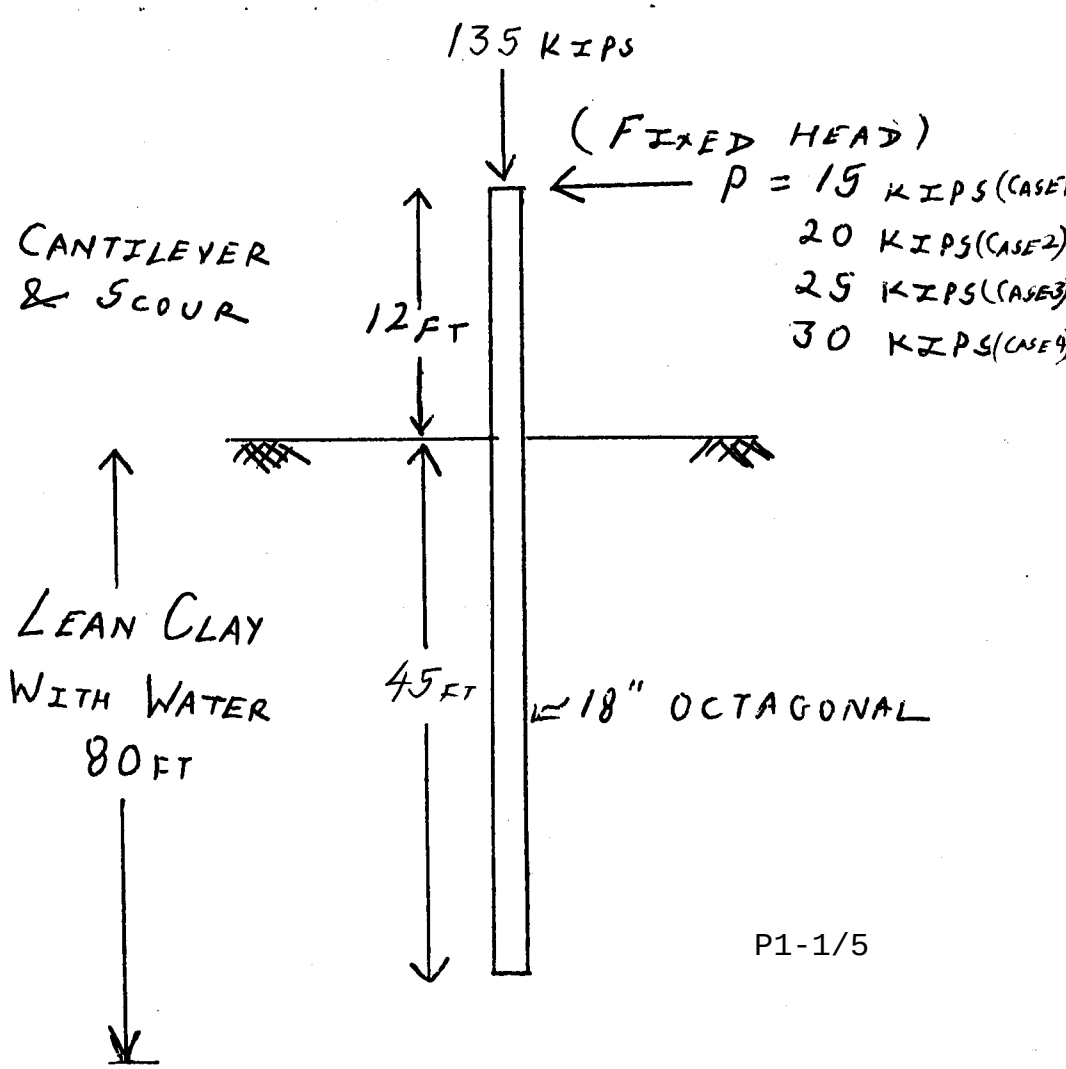

Lateral Deflection (in)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
Case 1
Case 2
Case 3
Case 4

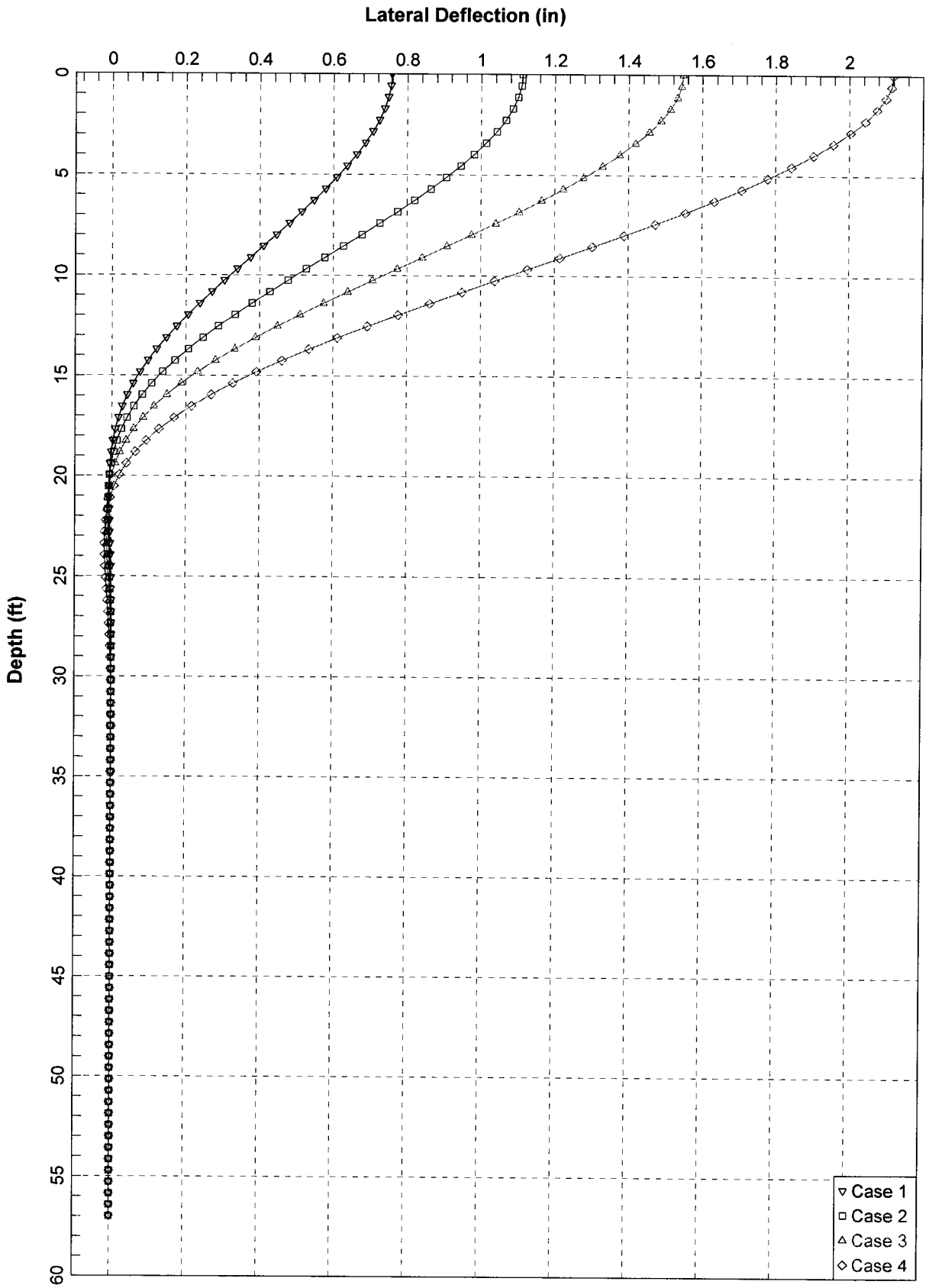

Shear Force (kips)
-30
-25
-20
-15
-10
-5
0
5
10
15
20
25
30
35
40
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
Case 1
Case 2
Case 3
Case 4

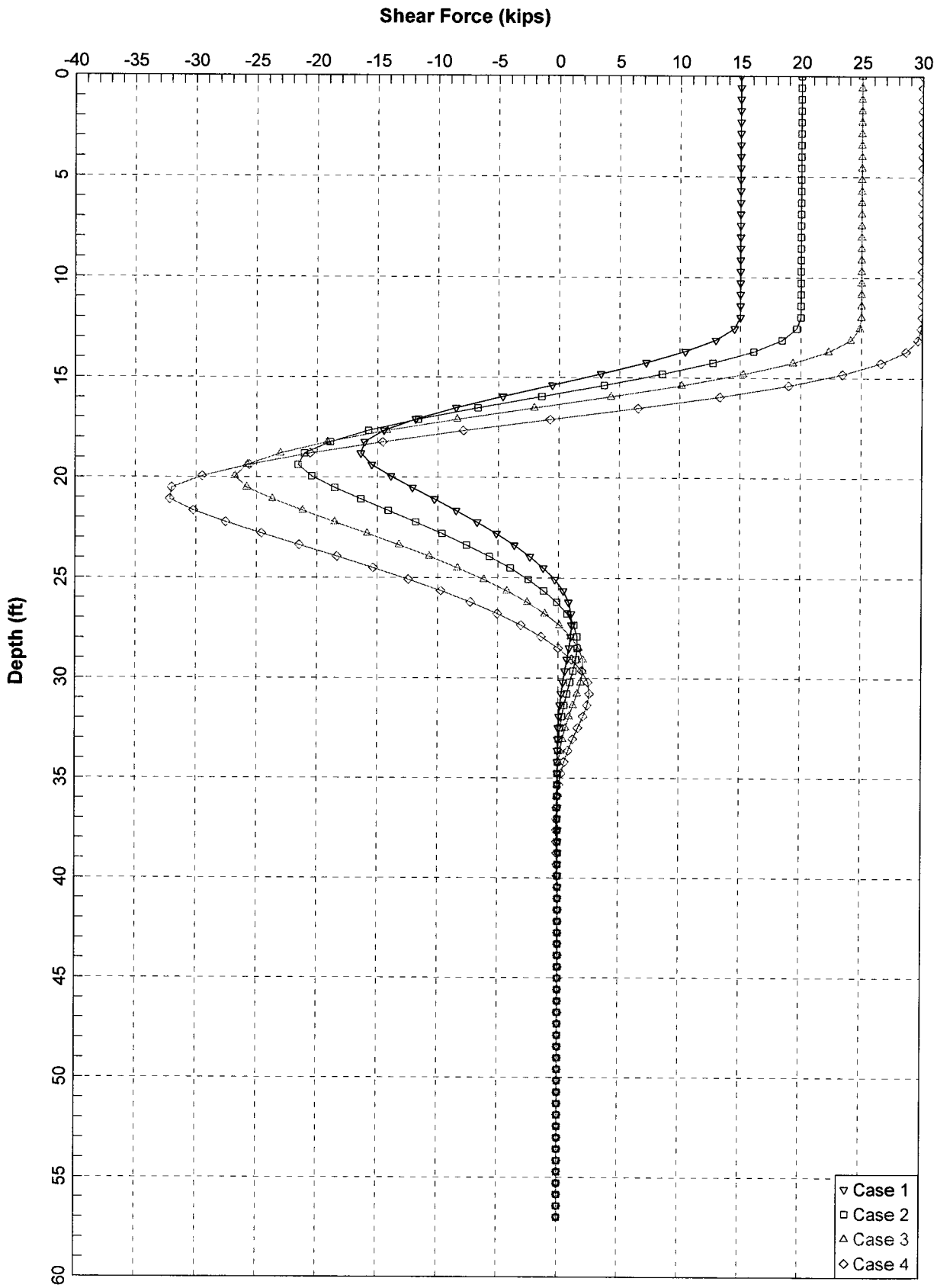

Unfactored Bending Moment (in-kips)
0
200
400
600
800
1000
1200
1400
1600
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
Case 1
Case 2
Case 3
Case 4

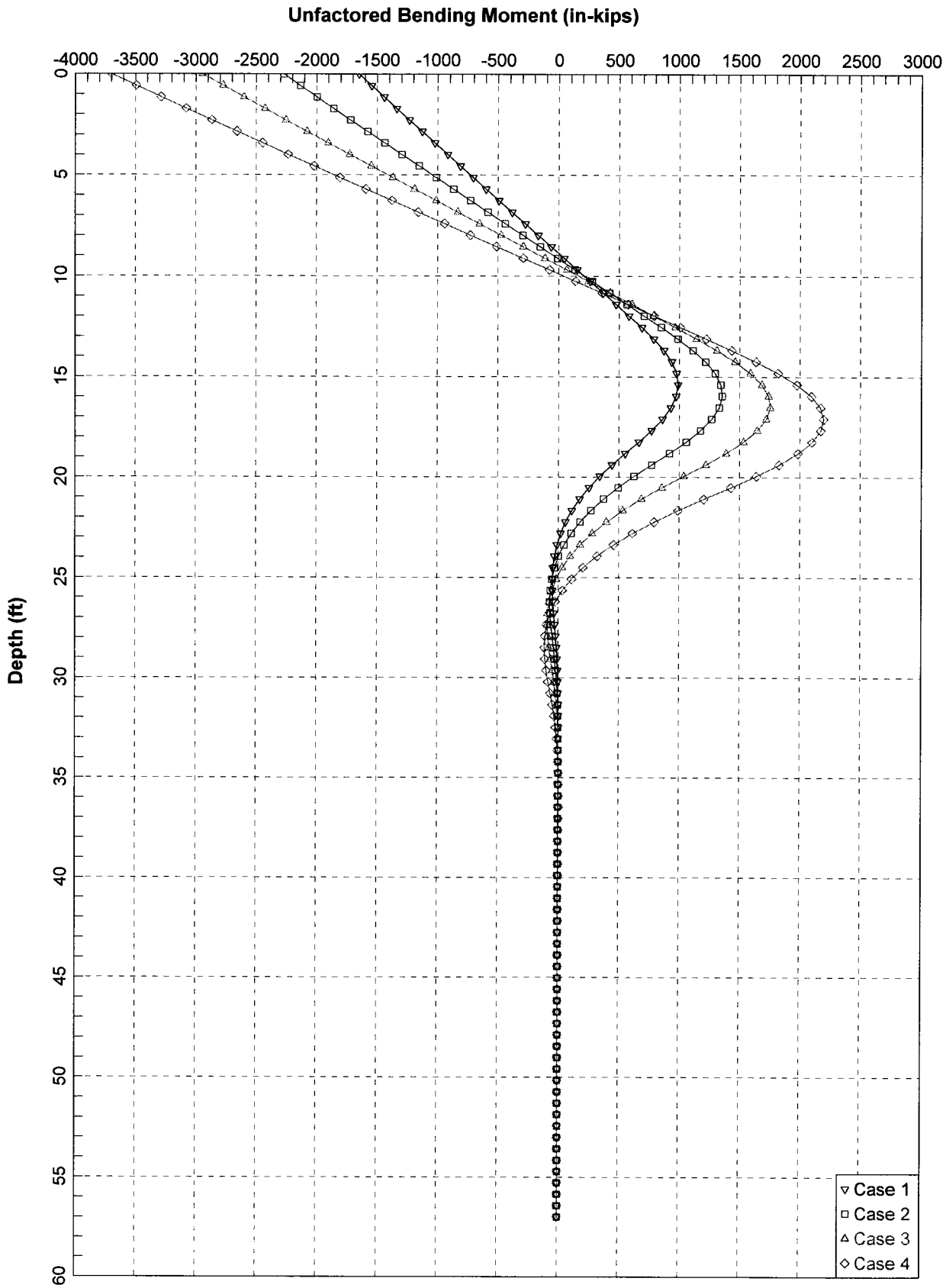

Mobilized Soil Reaction (lbs/in)
-600
-500
-400
-300
-200
-100
0
100
200
300
400
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
Case 1
Case 2
Case 3
Case 4

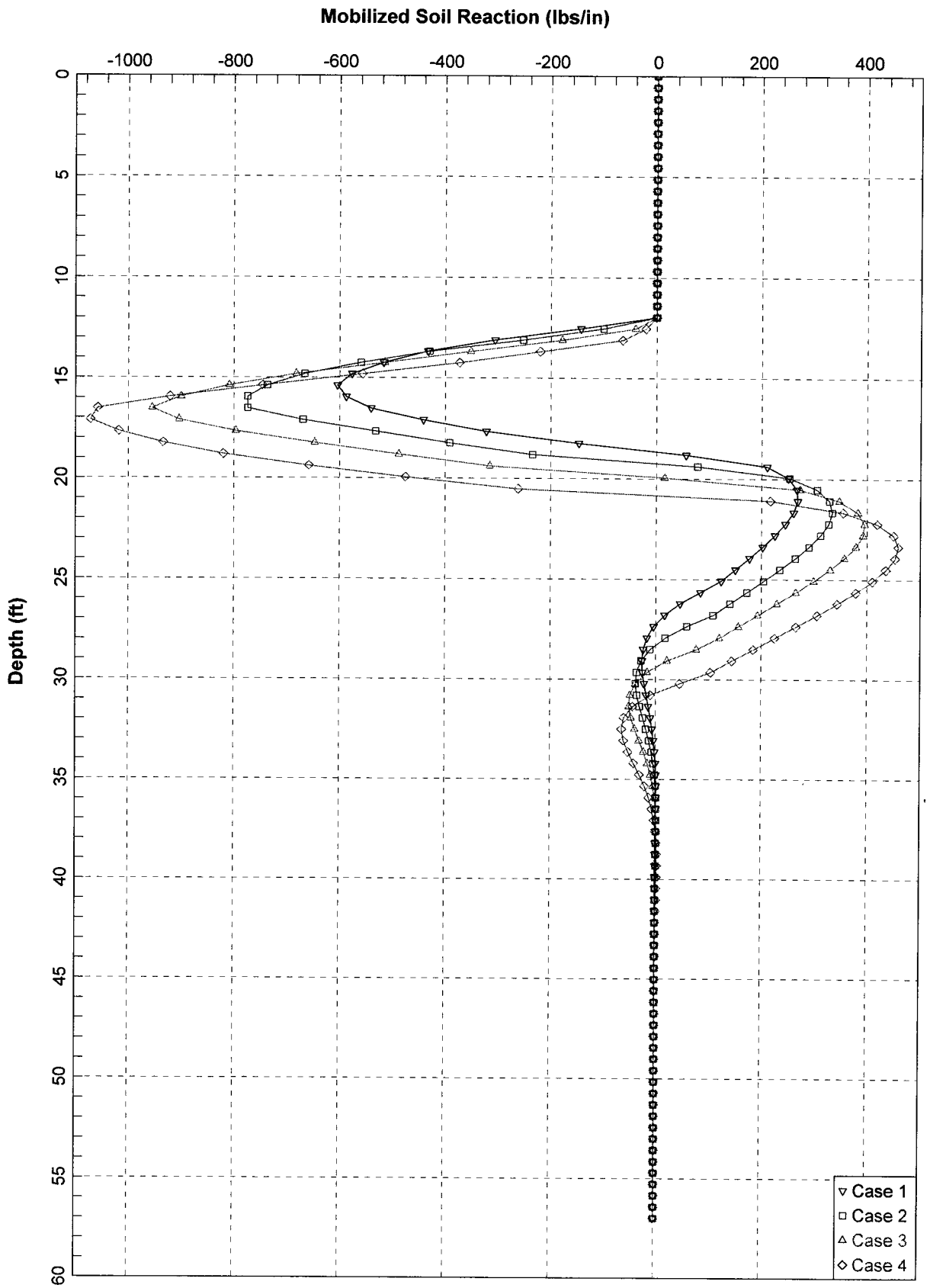
```
LPILEP5
New LPILE Plus 5.0 Data File
1       1       0       0       0       0
100     2       144     684     0
0       18      5705    255     3000000
684     18      5705    255     3000000
1       2       2       0       2
2       144     1000    1000    1000
144     0.034
1000    0.034
144     16.67   0       0.005   0
1000    16.67   0       0.005   0
144     0.6     1
1000    0.6     1
0       1       1
4
2       15000   0       135000
2       20000   0       135000
2       25000   0       135000
2       30000   0       135000
0
1       1       0
100     1E-5    100
```




Lateral Deflection (in)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
Case 1
Case 2
Case 3
Case 4

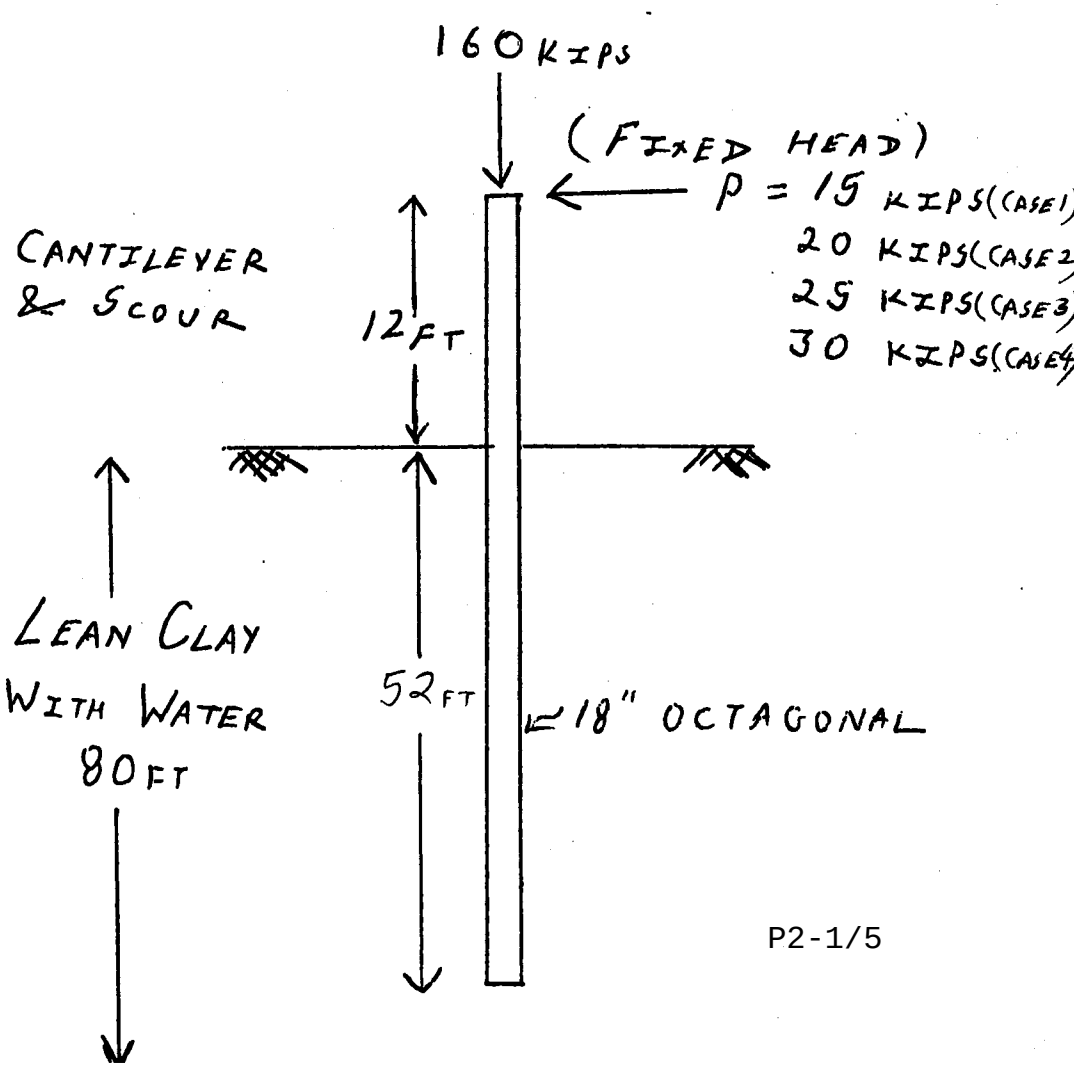

Shear Force (kips)
-40
-35
-30
-25
-20
-15
-10
-5
0
5
10
15
20
25
30
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
Case 1
Case 2
Case 3
Case 4

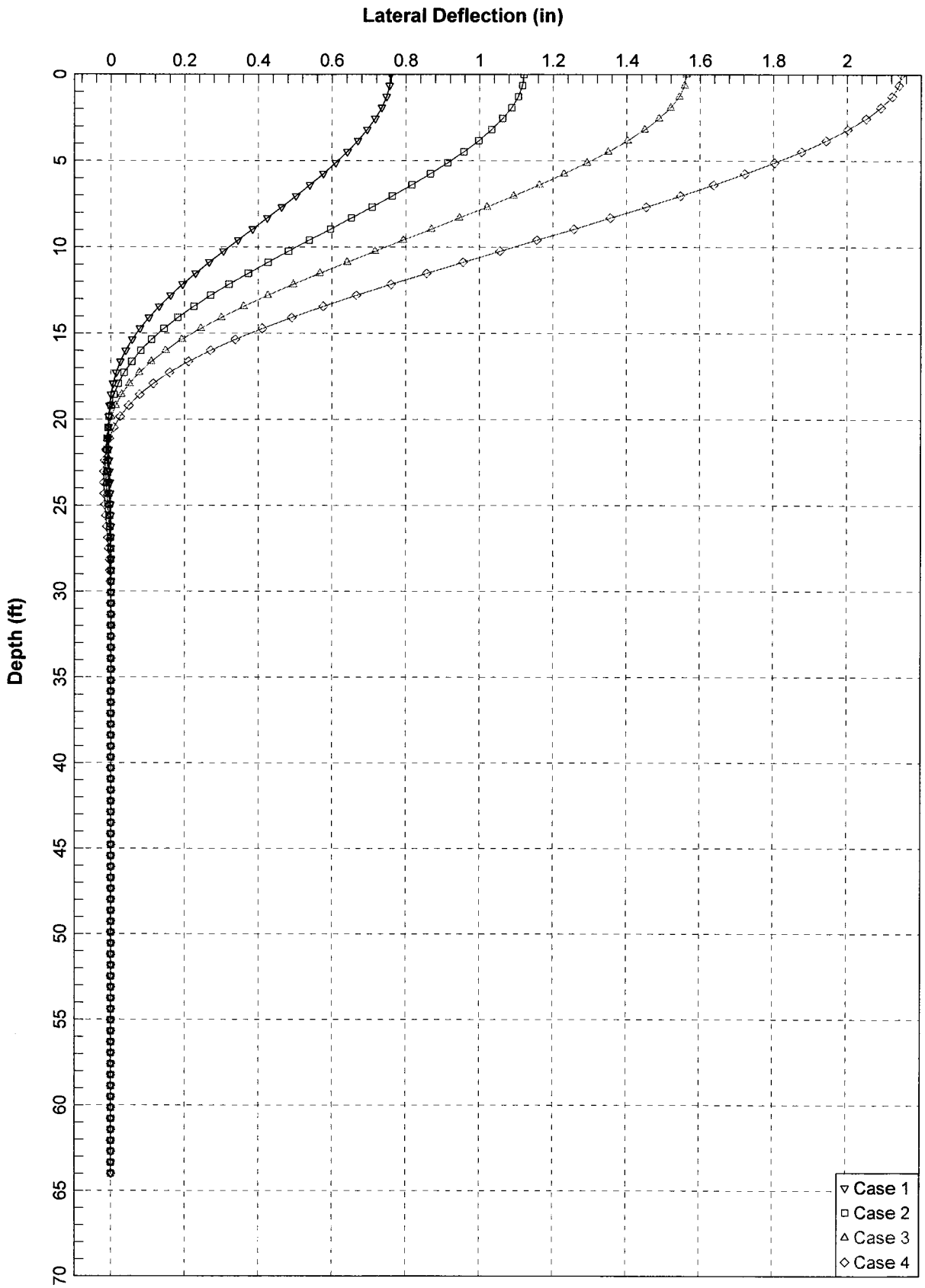

Unfactored Bending Moment (in-kips)
-4000
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
2000
2500
3000
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
Case 1
Case 2
Case 3
Case 4

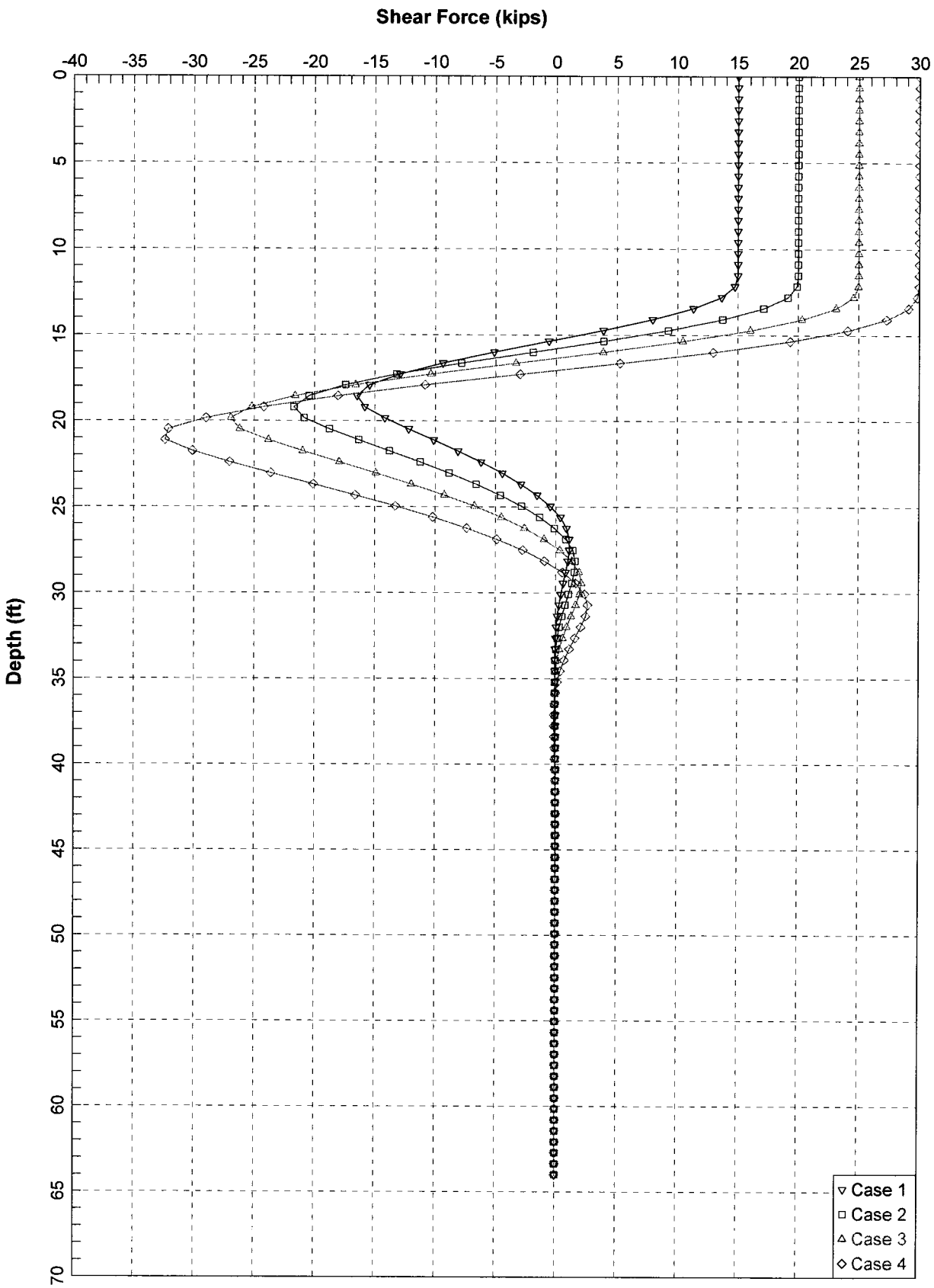

Mobilized Soil Reaction (lbs/in)
-1000
-800
-600
-400
-200
0
200
400
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
Case 1
Case 2
Case 3
Case 4

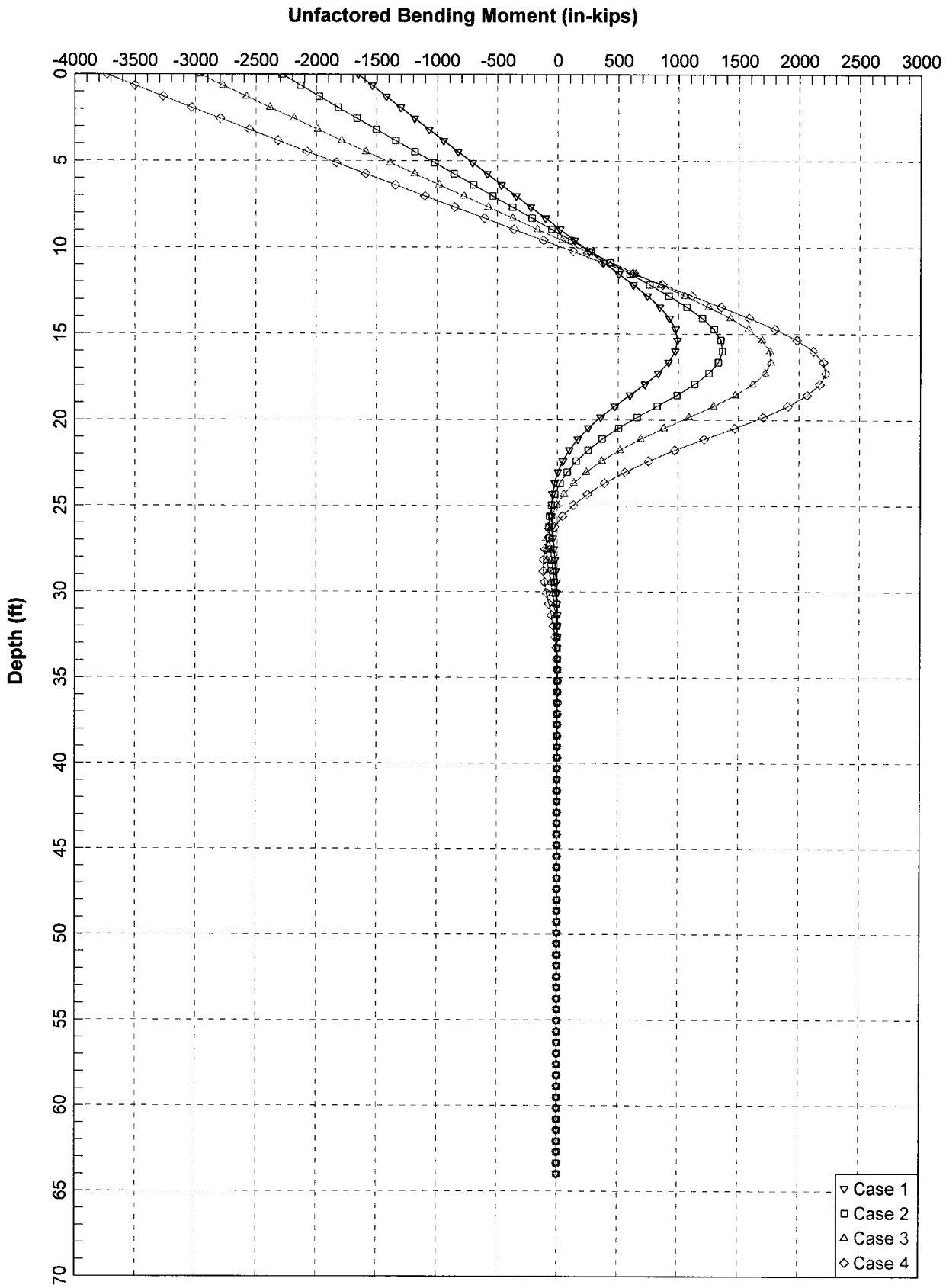
Pier-160.lpd

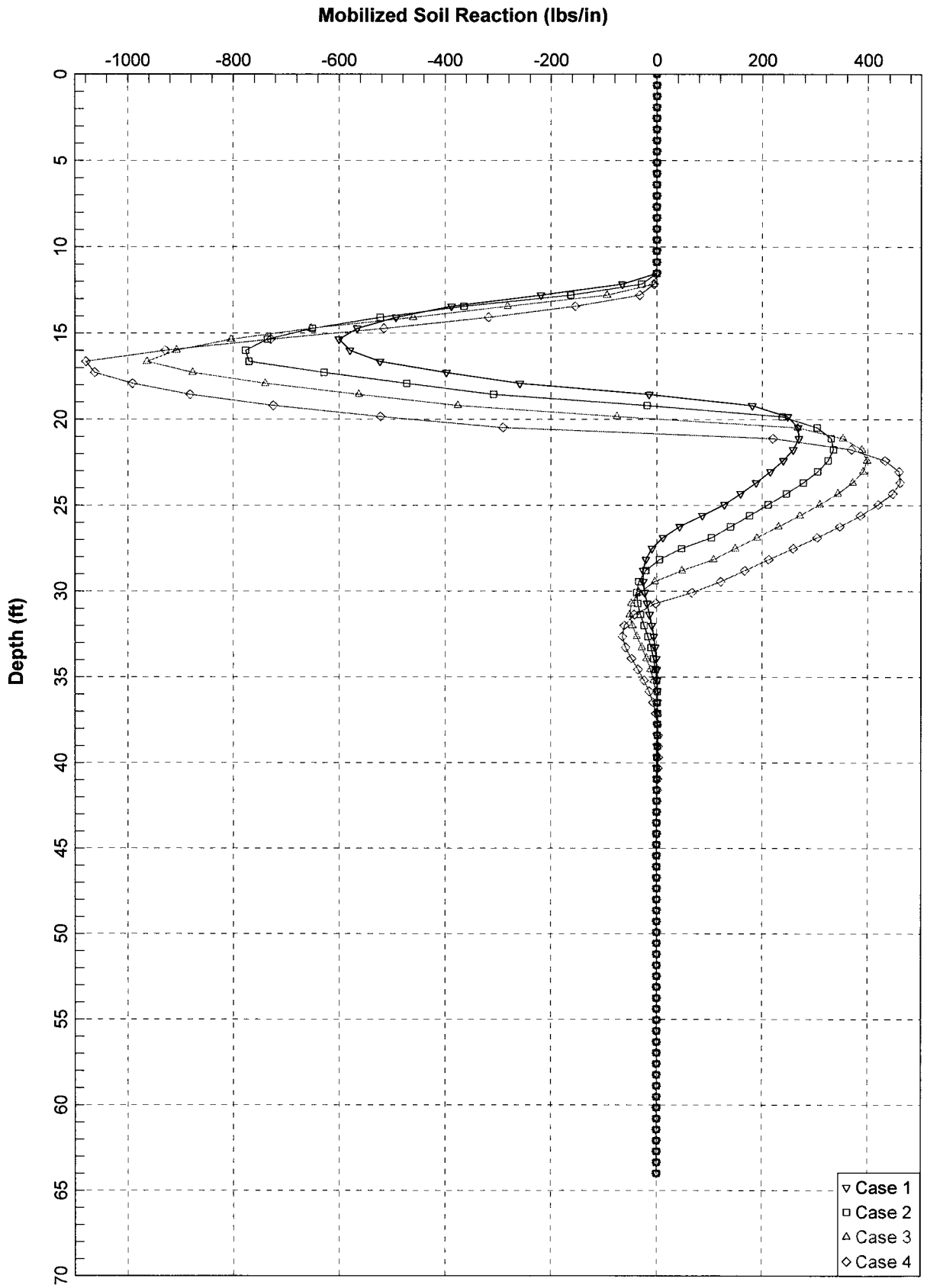
```
LPILEP5
New LPILE Plus 5.0 Data File
1       1       0       0       0       0
100     2       144     768     0
0       18      5705    255     3000000
768     18      5705    255     3000000
1       2       2       0       2
2       144     1000    1000    1000
144     0.034
1000    0.034
144     16.67   0       0.005   0
1000    16.67   0       0.005   0
144     0.6     1
1000    0.6     1
0       1       1
4
2       15000   0       160000
2       20000   0       160000
2       25000   0       160000
2       30000   0       160000
0
1       1       0
100     1E-5    100
```




Lateral Deflection (in)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
Case 1
Case 2
Case 3
Case 4


Shear Force (kips)
-40
-35
-30
-25
-20
-15
-10
-5
0
5
10
15
20
25
30
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
Case 1
Case 2
Case 3
Case 4


Unfactored Bending Moment (in-kips)
-4000
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
2000
2500
3000
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
Case 1
Case 2
Case 3
Case 4


Mobilized Soil Reaction (lbs/in)
-1000
-800
-600
-400
-200
0
200
400
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
Case 1
Case 2
Case 3
Case 4